THE UNIVERSITY OF ALBERTA

PRESTRESSED CONCRETE TEE BEAMS WITH LARGE WEB OPENINGS

by

© Jacques Germain Sauve

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled " PRESTRESSED CONCRETE TEE BEAMS WITH LARGE WEB OPENINGS", submitted by Jacques G. Sauvé in partial fulfilment of the requirements for the degree of Master of Science.

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ABSTRACT

This study is the first in a program to investigate the behaviour of and to develop design procedures for prestressed Tee beams containing large web openings. The test program was carried out in the Structural Engineering Laboratory of the University of Alberta under the guidance of Dr. J. Warwaruk.(*)

This test series was concerned with the behaviour of prestressed Tee beams containing large web openings and with the effect of varying the vertical and longitudinal reinforcements under different load positions.

Nine prestressed Tee beams were tested, eight of which contained 8in. by 16in. web openings. The beams all had an overall depth of 20in., a flange width of 20in. and a supported span of 20 feet. The variables considered consisted of varying two point loads, vertical shear reinforcement and supplementary longitudinal reinforcement.

The test results are presented in the form of tables, graphs and photographic plates.

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CHAPTER I

INTRODUCTION

The application of prestressed concrete Tee beams containing large web openings to multistorey structures is a very practical solution to the problem of wasted headroom, provided that the behaviour of such members is understood and that an efficient design is possible. The use of the contractor's jackhammer and of an uncalculated over abundance of reinforcing steel is neither a logical nor a practical solution. Until recent times, very little was known about the behaviour and design of prestressed and reinforced concrete members containing web openings.

For these reasons then, very recent experimental and analytical research has been directed toward the study of the behaviour and the design of such members.

Although limited in quantity, previous tests on prestressed and reinforced concrete beams containing web openings have indicated that the use of such members is possible, practical and economical if the load carrying capacity of these members can be maintained through various loading conditions.

This present test series is the first in a program at the

University of Alberta to investigate the behaviour of and to develop design procedures for prestressed concrete Tee beams containing large web openings. This series was mainly concerned with the behaviour of such beams and with the effect of varying the vertical and longitudinal reinforcement under different loading positions.

Nine beams were tested and the variables were, loading position, vertical or shear reinforcement and supplementary longitudinal reinforcement. Although the concrete strength and the effective prestress force were not considered as variables, small variations in these were unavoidable. The results are presented in terms of applied load and resulting deformations, with particular reference to the effect of the eight by sixteen inch web openings.

CHAPTER II

REVIEW OF PREVIOUS WORK

The use of random openings in webs of reinforced concrete beams to accomodate mechanical services is not a recent innovation; for example large openings are found in reinforced concrete Vierendeel trusses. More recently, new developments in the field of reinforced and prestressed concrete structures require the engineer to design openings to accomodate services, but to date, only a very limited amount of research has been performed and published on beams having large web openings.

2.1 Prestressed Tee beams with large web openings

Ragan and Warwaruk[1] conducted tests on prestressed Tee beams containing web openings in the Structural Engineering Laboratory of the University of Alberta. In this investigation, four model beams were tested in the laboratory and two full size beams were tested in the field. The design of the two full size members was based on the results obtained from the model beams.

Some of their observations and conclusions are:

In the model beams;

a) cracking extended vertically downward from approximately the center point of the web openings; hence it was concluded in

the full size beams to distribute the prestressing strands almost evenly across the vertical section of the lower web.

b) severe cracking at the connection of the "post" and flange led to the provision of steel in the posts at 1.3% of the horizontal area of these posts.

c) all failures were due to inclined cracking in the lower web and always in the half span which had the least amount of web reinforcement; hence the full size beams were provided with U-stirrups in the lower web spaced at 6 to 12 in..

d) the mode of failure of all beams with openings was by the formation of mechanisms. None of the beams, neither model nor full size failed in a flexural manner.

2.2 Reinforced concrete Tee beams with a web opening

Lorentsen[2], of the Royal Institute of Technology in Stockholm, conducted analytical and experimental research on reinforced concrete girders having a single web opening. Four beams were tested under different loading conditions.

His tests confirmed in general the behaviour pattern predicted by the elastic theory and also showed that there was a reserve of capacity available due to the redistribution of moments between the stressed sections located at the edges of the opening. Lorentsen also showed that for statically loaded structures, satisfactory structural capacity can be achieved if the sections near a hole are designed to resist the normal and bending forces. He pointed out that in simply supported beams, there are two quantities which make the location of a hole near the midspan desirable;

(i) the magnitude of the flange moment,

(ii) the shear force.

In simply supported beams the flange moment is small near a support and the shear force is large. The combination of these two quantities yields high principal tensile stresses in the concrete in regions away from the midspan.

An important conclusion which he made, is that in the region of a hole, the members should be overdesigned with respect to the crushing of the concrete. This guards against the occurrence of a plastic condition at working loads. This condition is not desirable since the moment may change in sign in this region depending on the position of the load. He also concludes that the principles presented can be applied to cases where more than one hole exists and to long walls containing door openings.

2.3 Rectangular reinforced concrete beams with large web openings

Nasser, Acavalos and Daniel[3], of the University of Saskatchewan, tested nine rectangular beams containing web openings. They were able to verify that large openings behave similar to a Vierendeel panel, that contraflexure points exists at the approximate midspan of the cross members, that the diagonal force concentration at the corners of an opening is twice the simple shear force, and that adequately reinforced large openings do not reduce the ultimate capacity of a beam, but reduce the stiffness and hence increase deflections.

CHAPTER III

RESEARCH PROGRAM

The present test series is the first in a program to investigate the behaviour of and to develop design procedures for prestressed Tee-beams containing large web openings. This series was concerned with the study of the behaviour of such members and with the effect of varying the vertical and longitudinal reinforcement under different loading positions.

Nine prestressed Tee-beams were tested. All of the beams were 24 feet long, had a supported span of 20 feet and had an overall depth of 20 inches. The first beam was a control beam and contained no holes, while the other eight beams each contained eight 16 " X 8" holes spaced at 2 feet on center . The predesignated variables consisted of ; varying two point load positions, vertical shear reinforcement and supplementary longitudinal reinforcement in the shear spans. Beam sections, elevations, load positions and plan reinforcement details are shown in Figs. 3.1, 3.2, 3.3 and 3.4 respectively. The effective prestress force, developed by 4-3/8" 7 wire strands and the concrete strength were not variables although small variations in these were unavoidable.

The vertical or shear reinforcement used throughout the fabrication of the beams consisted of #3 stirrups, prebent to a spe-

cified shape by the supplier. The supplementary longitudinal reinforcement consisted of #3 or #4 deformed reinforcing bars. The steel in the control beam was designed in accordance with A.C.I. standard (318-63 & 71)[4] ; see Appendix C for further details.

Special equipment was designed for the prestressing operation, for the concreting and for the testing operation. This equipment included: all steel prestressing abutments, slip type of formwork including styrofoam blocks for shaping the openings, and steel beam seats. The seats were fitted on the beams at the support points and were then bolted in place; Fig. 3.5 depicts a typical seat.

All beams were cast using High Early Strength cement. The prestressing strands of beams #1 to 6 were cut after six days of moist curing, while those of beams #7 to 9 were cut after five days. Following the release of prestress the beams were stored in laboratory atmosphere for varying periods prior to testing. Testing took place at times varying from 17 to 28 days after casting. Concrete test cylinders were molded from the batch mixes used and were tested for compressive strength and tensile splitting strength on the same day as the beams.

All nine beams were instrumented in the same fashion. The instrumentation consisted of electrical resistance strain gages mounted on the vertical and longitudinal reinforcement at critical points, survey level readings at centerline and the one-third points

for deflection measurements, and Demec points fixed to the concrete at centerline for strain distribution measurements. Electrical resistance strain gages were waterproofed by applying an epoxy type paint cover. Level readings were taken off scales, graduated to 0.01 in., which were suspended from the lower portion of the beam. The primary use of the Demec points was for the measurment of initial shortenings and time losses in 0.0001 in.. In addition, the strain between the Demec points were taken throughout actual testing. Figure 3.6 shows typical instrumentation details.

The beams were loaded using an Amsler loading apparatus by applying two equal point loads on the longitudinal centerline of the top flange. The loads were applied at various predetermined positions and always at the center of the 8 in. vertical posts. Figure 3.3 shows the different load positions and Fig. 3.4 shows the loading positions for particular beams. A steel plate was placed under each jack to ditribute the load from the jack to the beam;Fig. 3.7 illustrates a typical test set up.

Sufficient load increments were applied in order that behaviour could be well recorded; increments of 1.0 kip were used up to cracking and 0.5 kip from cracking to failure. At the end of each increment readings were taken of the electrical resistance strain gages, of the deflections and of the Demec points. All visible cracking was observed and recorded on the surface of the concrete. To provide a good record of cracking behaviour, photographs were taken before and after failure.

TABLE 3.1

TEST BEAM REINFORCEMENT DETAILS

BEAM No.	STIRRUP SPACING (inches)	AREA (in^2)	EQUIVALENT GROUPING OF #3 STIRRUPS (stirrups/post)	AREA (in^2)	LONGI. REINFORCEMENT LOWER WEB (in^2)	UPPER WEB (in^2)	LOAD POSITIONS FROM SUPPORTS (inches)
1	12	.22	-	-	.22	.66	72
2	12	.22	2	.44	.22	.66	72
3	12	.22	2	.44	.22	.66	96
4	12	.22	2	.44	.22	.66	48
5	8	.33	3	.66	.22	.66	72
6	8	.33	3	.66	.22	.66	48
7	8	.33	3	.66	.40	1.11	72
8	8	.33	3	.66	.40	1.11	48
9*	8	.33	3	.66	.40	1.11	72

* Beam #9 also had 2 #3 diagonal U-stirrups in the lower webs of the shear spans

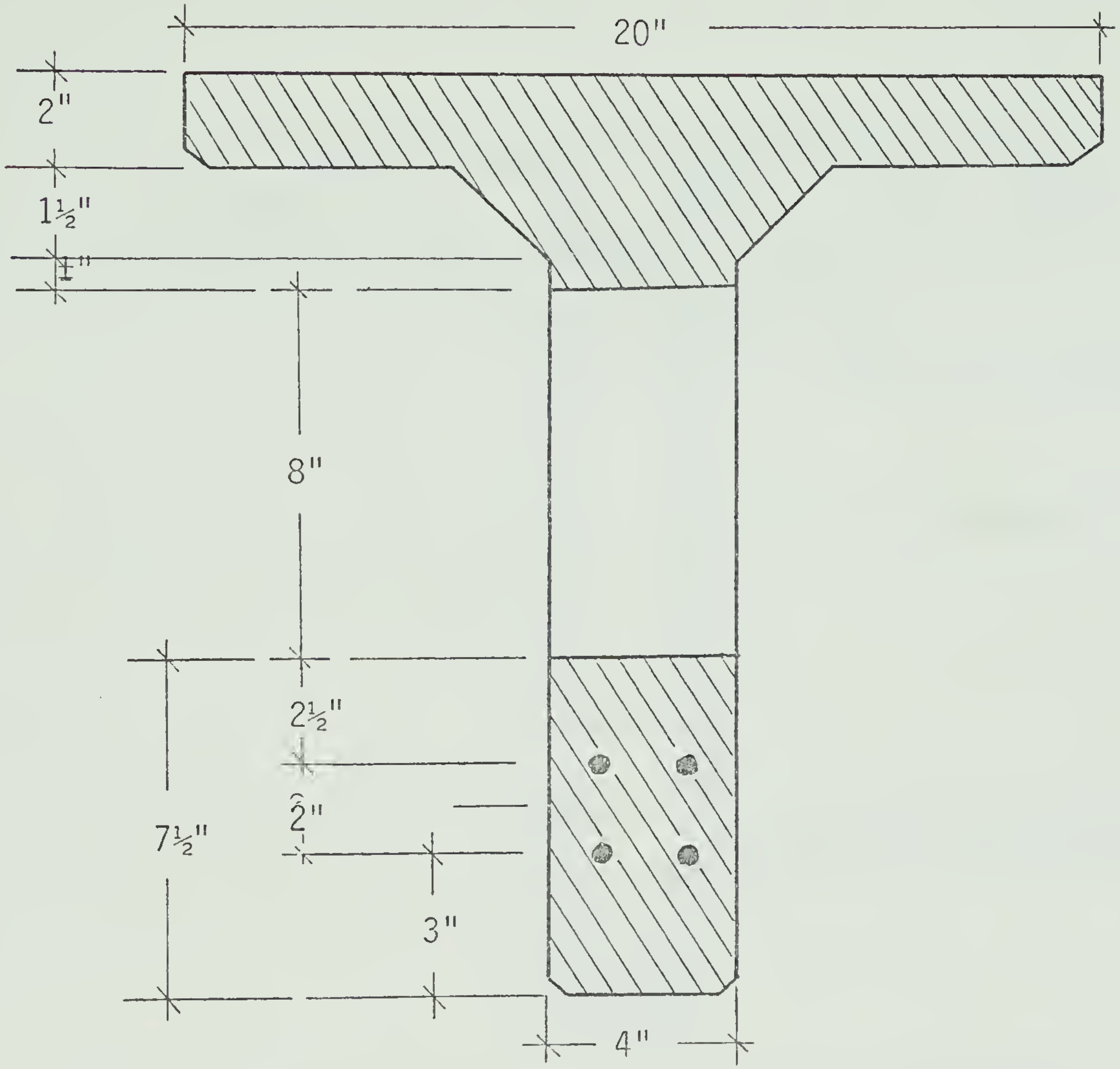


FIGURE 3.1 TYPICAL CONCRETE X-SECTION AT AN OPENING

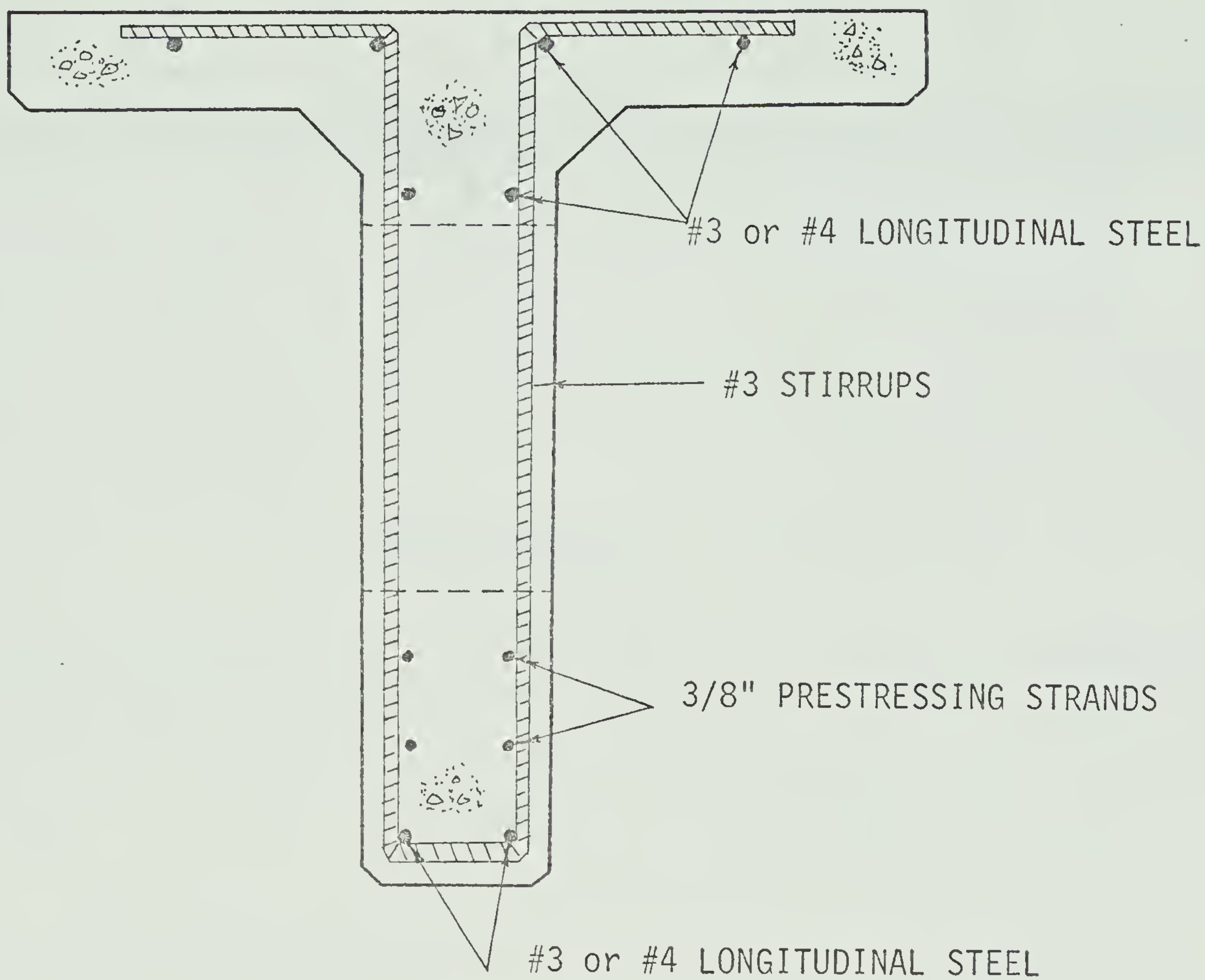
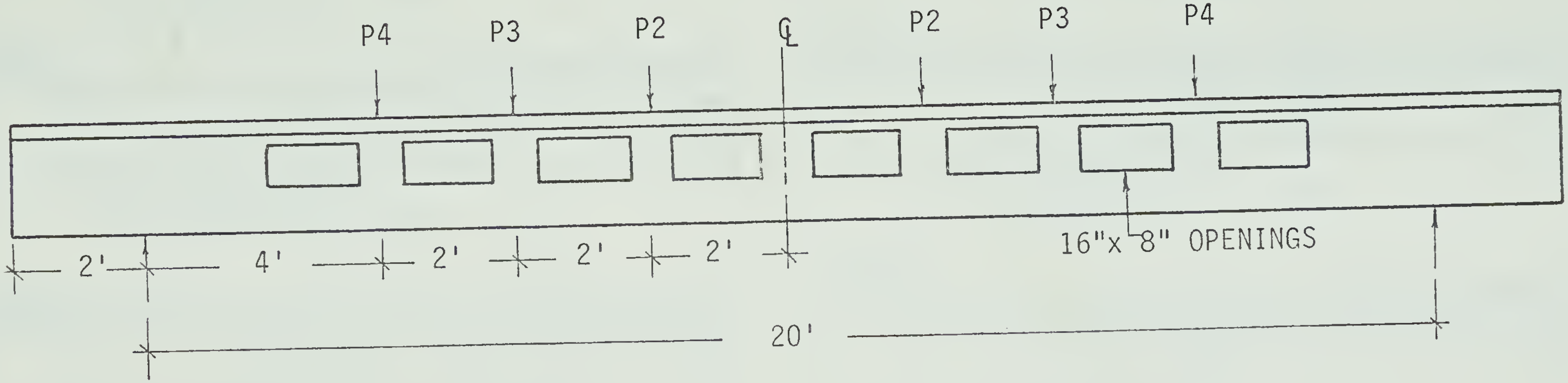


FIGURE 3.2 TYPICAL BEAM REINFORCEMENT DETAIL



LOAD POSITIONS

FIGURE 3.3 POSITIONS OF APPLIED TWO POINT LOADS

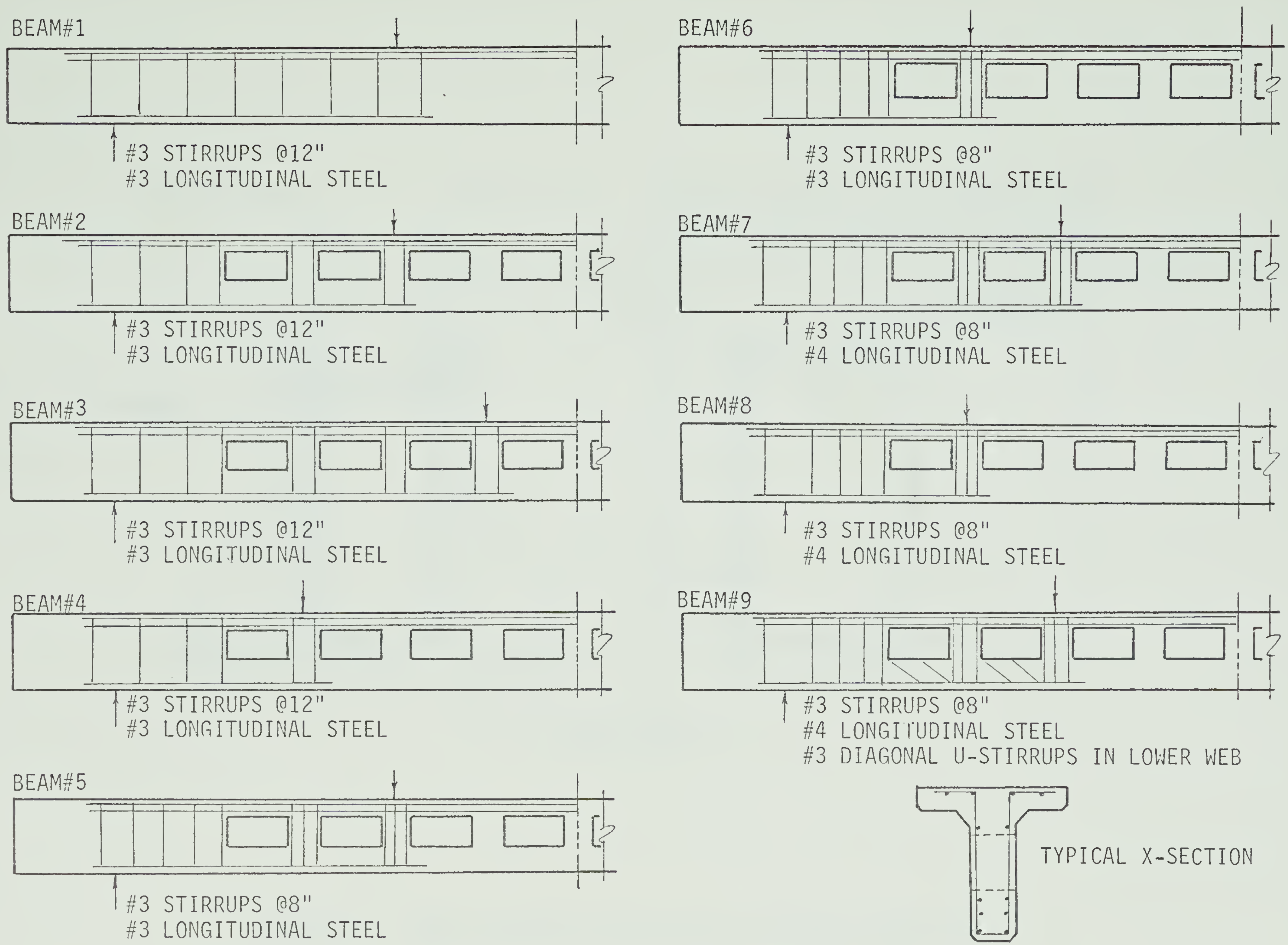


FIGURE 3.4 TEST BEAMS AND REINFORCEMENT DETAILS

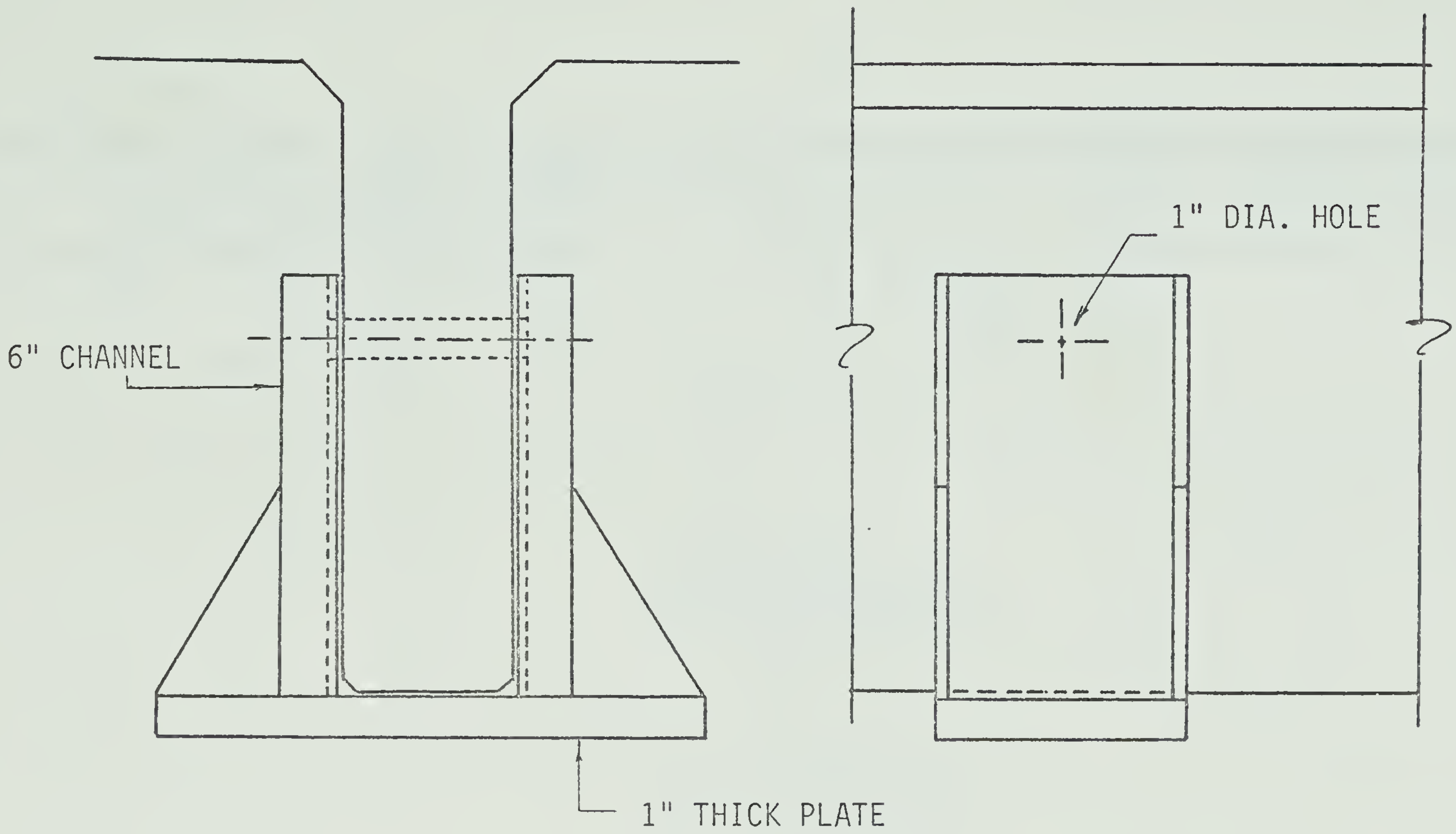


FIGURE 3.5 FRONT AND SIDE ELEVATIONS OF TYPICAL BEAM SEATS

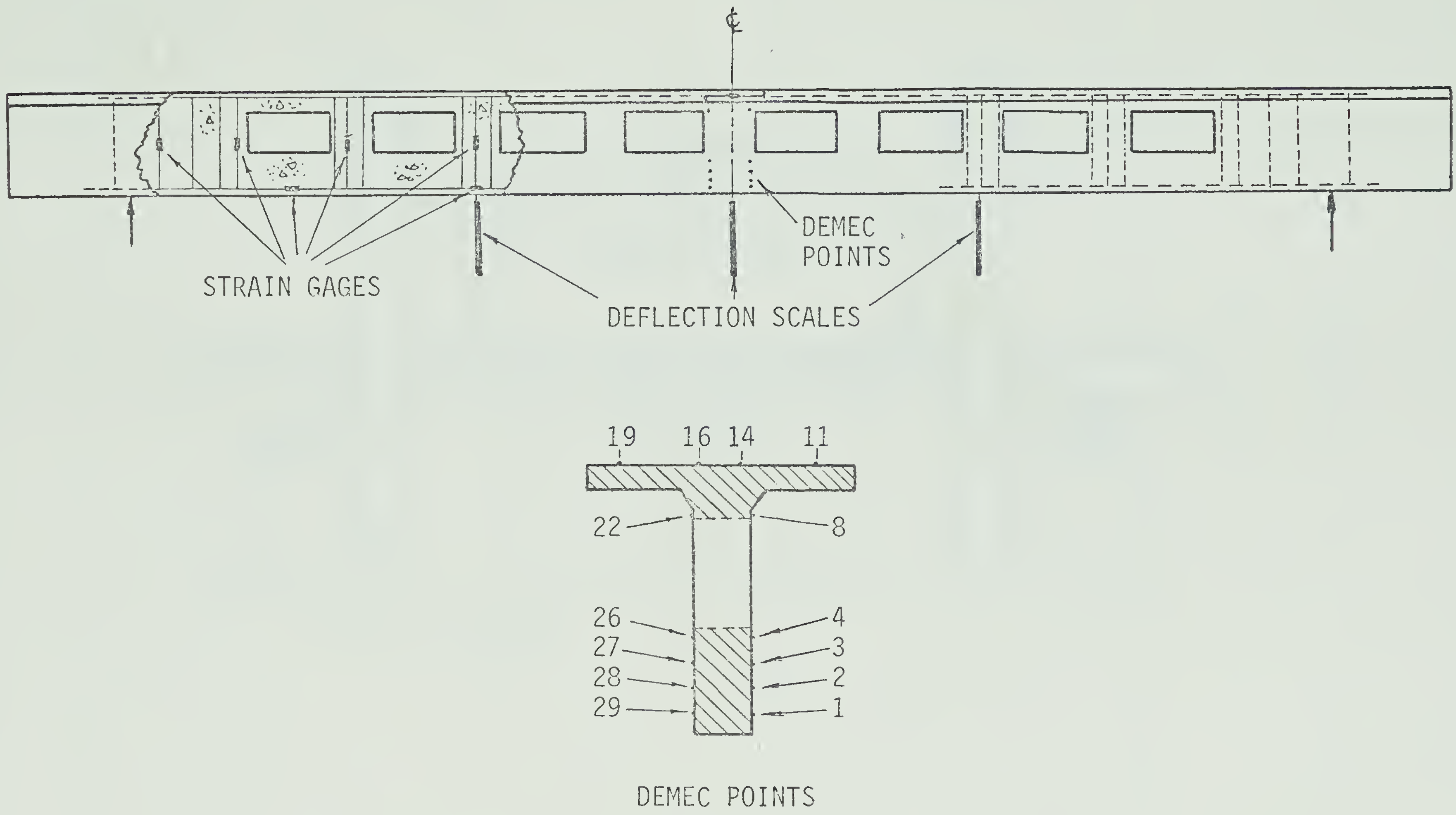


FIGURE 3.6 BEAM INSTRUMENTATION DETAILS AND LOCATIONS

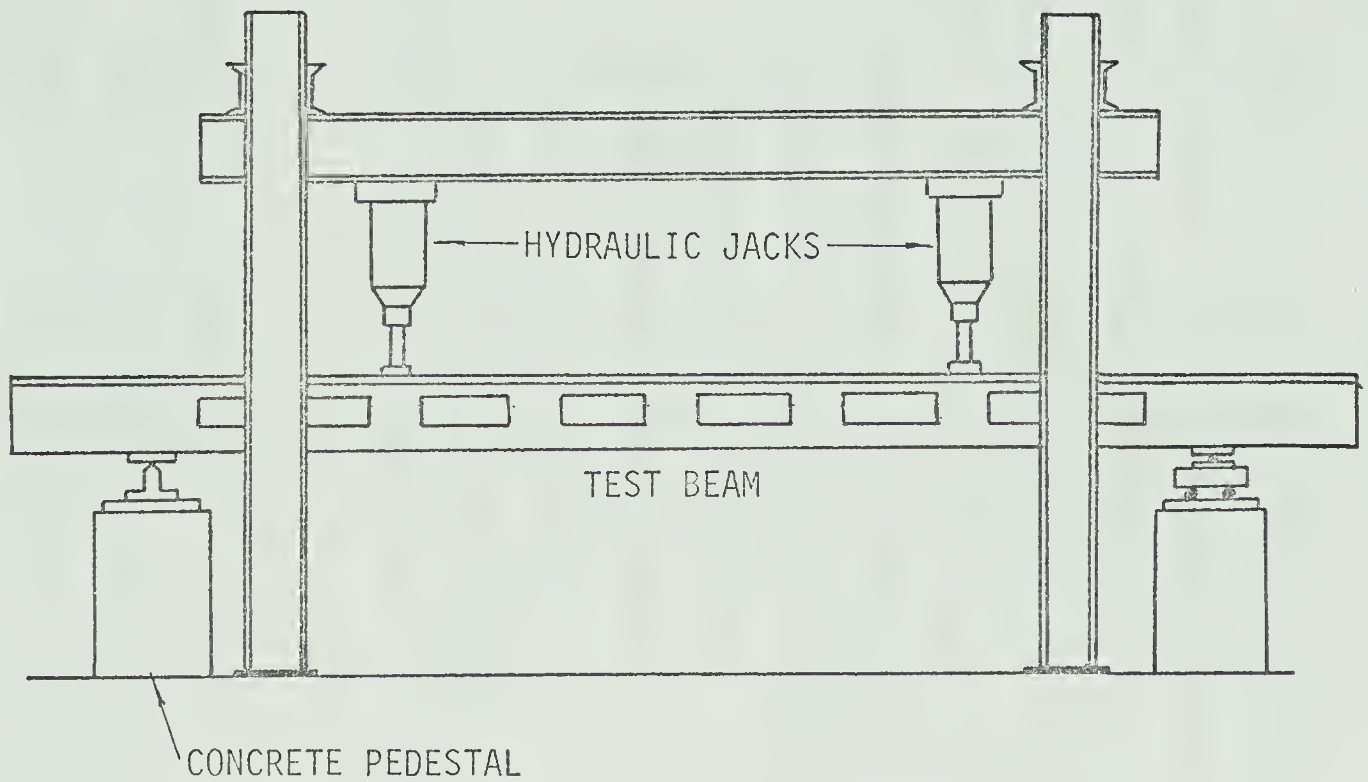


FIGURE 3.7 LOADING FRAME AND TEST SET-UP

CHAPTER IV

TEST RESULTS

The graphical and photographic results presented in this chapter were obtained both directly and indirectly from the measurements taken during the actual testing of the nine prestressed Tee beams. The measurements taken were those of electrical resistance strain gages, mechanical Demec strain gage, survey level and Amsler hydraulic loading apparatus. The instrumentation of the beams is depicted in CHAPTER III. The graphical results consist of:

1) Load-deflection relationships
2) Strain distribution diagrams
3) Moment-strain relationships
4) Load-stirrup force relationships.

The photographic results are comprised of cracking and failure patterns.

4.1 Load-deflection relationships

The deflections used in Figs.4.1 (a) and (b) were those obtained directly from the survey level readings. Beam deflections were taken at the one-third points and at the centerline. Applied loads were read directly from the Amsler loading apparatus scale.

Figures 4.1 (a) and (b) present the load-centerline deflec-

tion curves for the beams grouped according to load position 3 and load positions 4,2 respectively. It should be noted that the load-deflection curve for beam #1 is not complete as this beam was unloaded and reloaded to failure the next day. Individual load-deflection diagrams for the beams are presented in Appendix B.

4.2 <u>Strain distributions over the depth of section</u>

The strains, over the depth of the beam at midspan, depicted in this section were those resulting from the change in length between the Demec points mounted on the test specimens. The readings were taken at those points shown in the instrumentation diagram, Fig. 3.6. Strains were plotted versus depth for particular load values for each beam. The plots for each beam were in turn grouped according to the load positions and always related to the control beam plots. Figure 4.2 (a) shows these relationships for beams of load group 3 and Fig. 4.2 (b) for those of beam #1 and load groups 2 and 4. The last strain diagram for most beams are those of the beams' penultimate load.

4.3 <u>Moment-reinforcement strain relationships</u>

The reinforcement strains were measured directly from the electrical resistance strain gages mounted on the supplementary reinforcement. Locations of the gages for particular beams are presented in Appendix B, along with the direct readings. The moment values used were those of the applied load times the distance from support to load point. Figure 4.3 (a) presents moment versus tensile strain in the bottom supplementary reinforcement under the point of load and

Fig. 4.3 (b) shows moment versus compressive strain in the top reinforcement at the centerline. The results of beams #2,7 and 8 in Fig. 4.3 (a) and beam #7 in Fig. 4.3 (b) are not plotted as the strain gages concerned were not operative.

4.4 Applied load-stirrup force relationships

The results for this group of relationships consisted of plotting the applied load versus the stirrup force for each increment of load. Applied loads were read directly from the Amsler load apparatus while the stirrup forces were obtained indirectly from measurement of the steel strains. Locations of the strain gages for particular beams are presented in Appendix B, along with the direct strain readings. The stirrup forces were scaled off the stirrup stress-strain curve which was obtained from direct tensile tests made using a Baldwin Testing Machine. Plotted results were grouped, as shown in Figs. 4.4 (a), 4.4 (b), and 4.4 (c) respectively, according to the shear section considered. Strain readings for shear section 4 were not available due to malfunction of the respective electrical resistance strain gages.

4.5 Illustrative cracking and failure results

This section consists of photographic plates of the cracking and failure patterns of beams #1 to 9 inclusive. The plates are grouped with respect to the loading positions.

4.6 Summary of test results

Table 4.1 presents a summary of test results indicating load position, load at failure, moment at failure, type of failure,

amounts and type of reinforcement and concrete strengths.

TABLE 4.1

A SUMMARY OF THE TEST RESULTS

BEAM No.	LOAD POSITION FROM SUPPORT (INCHES)	LOAD AT FAILURE (KIPS)	MOMENT AT FAILURE (KIP-INCH)	TYPE OF FAILURE	REINFORCEMENT IN SHEAR SPAN		f'_c (PSI)
					LONGITUDINAL	VERTICAL	
1	72	20.3	1462	FLEXURAL	#3	@12"	4686
2	72	13.5	973	SHEAR	#3	2*	4913
3	96	13.9	1335	SHEAR	#3	2*	4797
4	48	17.0	817	SHEAR	#3	2*	4219
5	72	16.5	1189	SHEAR	#3	3*	4922
6	48	19.5	935	SHEAR	#3	3*	4850
7	72	15.5	1120	SHEAR	#4	3*	4870
8	48	20.05	860	SHEAR	#4	3*	4823
9 **	72	17.5	1260	SHEAR	#4	3**	5152

* INDICATES No. OF STIRRUPS PER POST

** BEAM #9 ALSO HAD 2 #3 INCLINED STIRRUPS IN THE LOWER WEBS SUBJECTED TO SHEAR

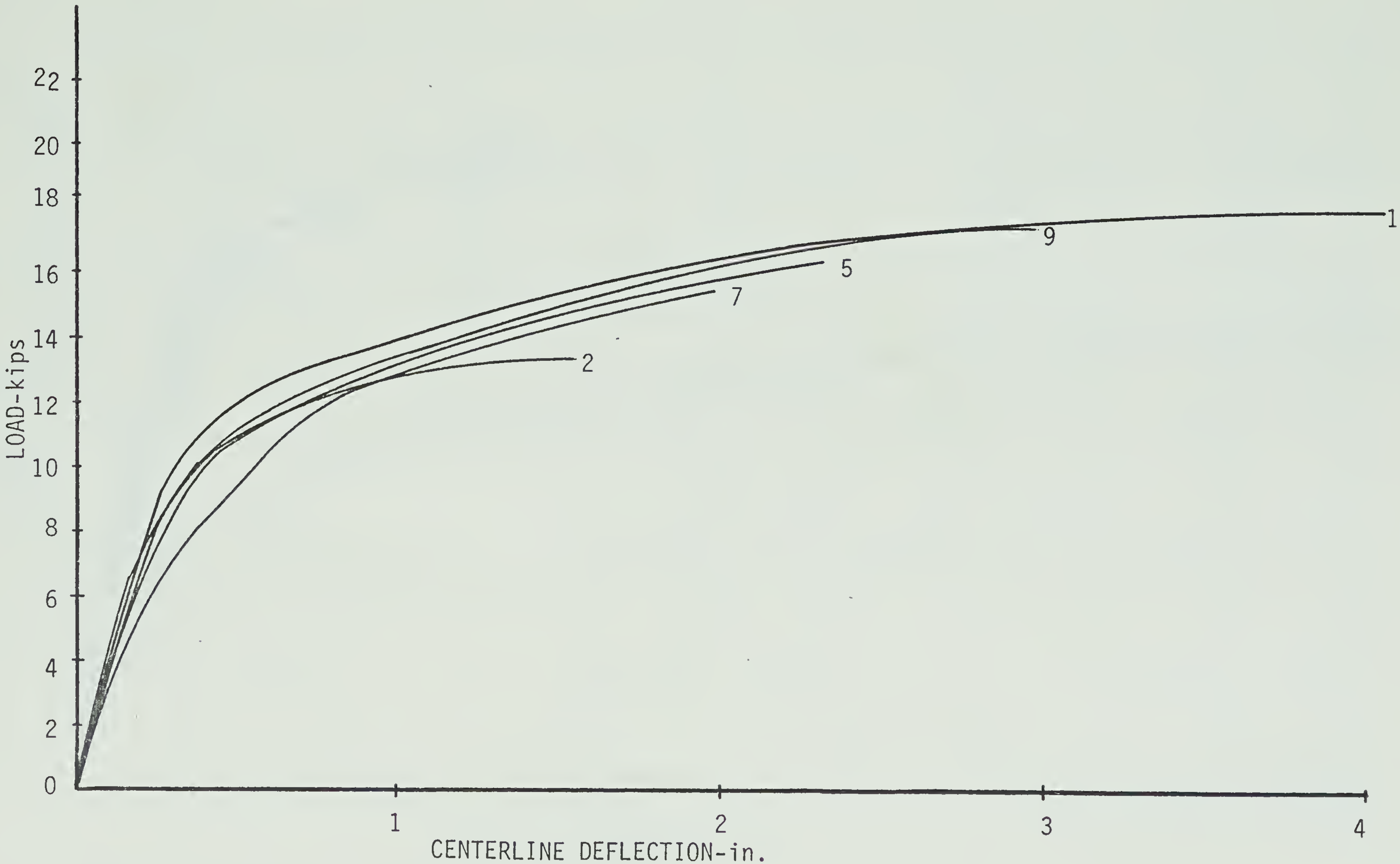


FIGURE 4.1 (a) LOAD-DEFLECTION DIAGRAM FOR BEAMS OF LOAD GROUP 3

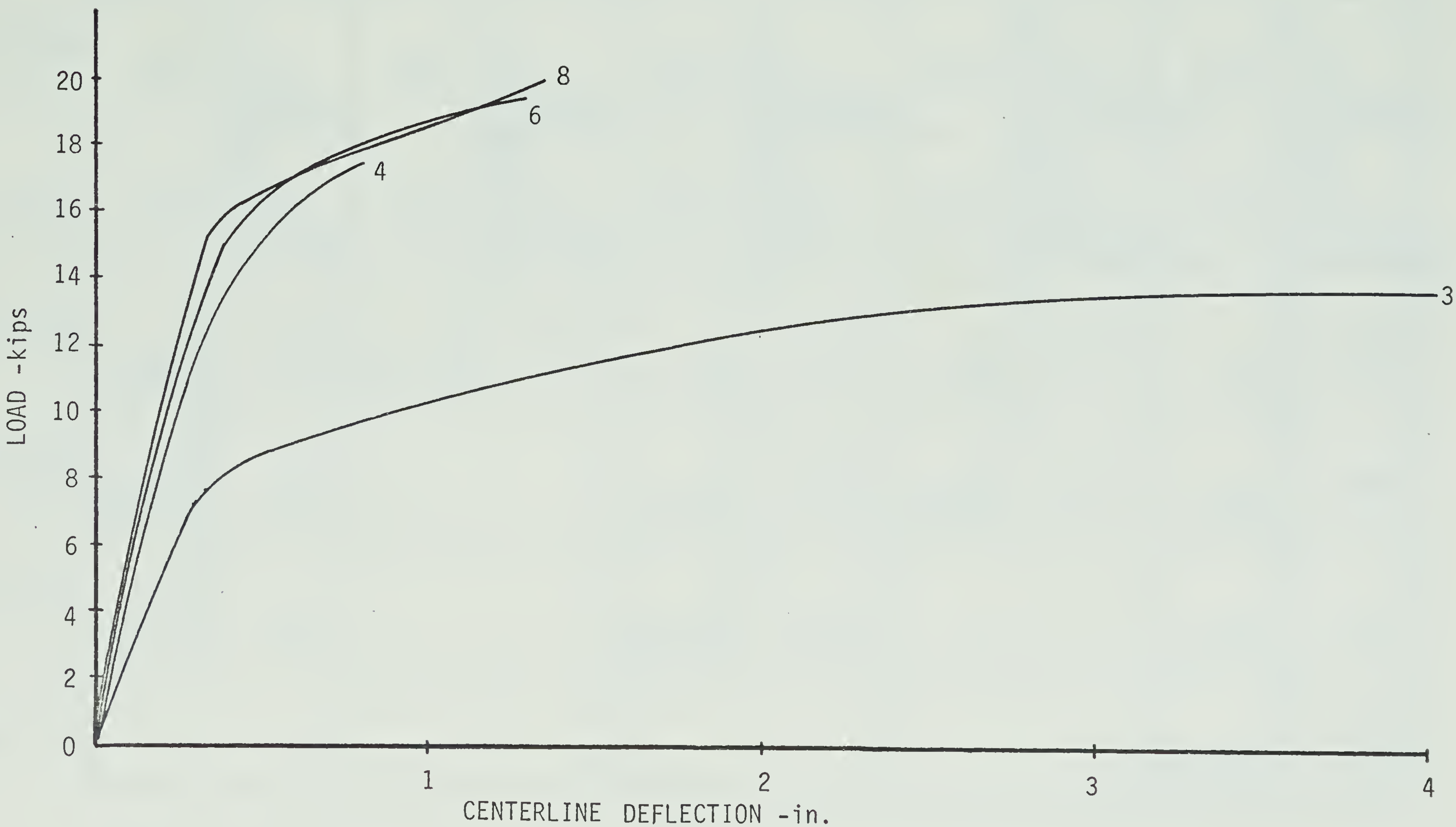


FIGURE 4.1 (b) LOAD-DEFLECTION DIAGRAM FOR BEAMS OF LOAD GROUPS 2&4

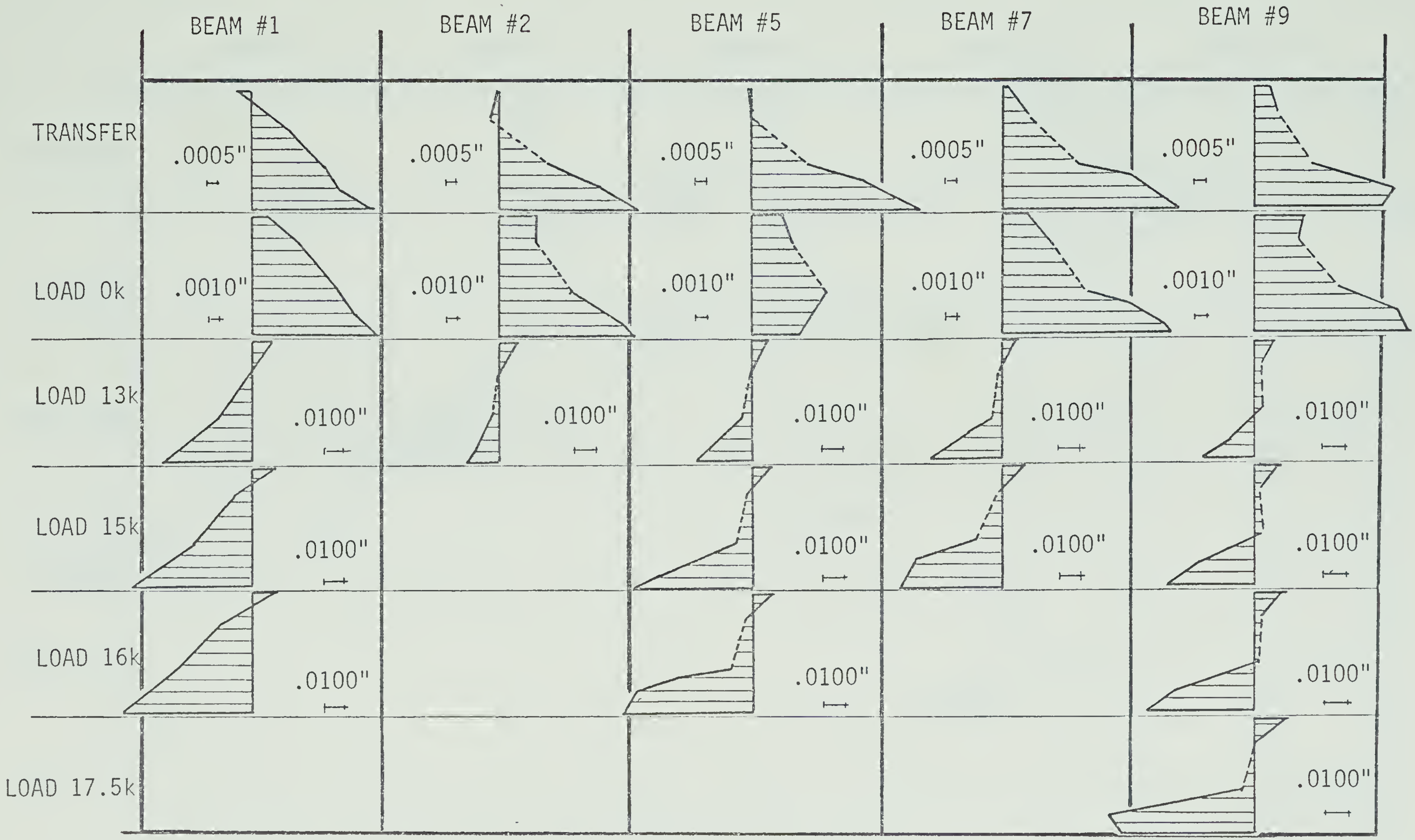


FIGURE 4.2 (a) STRAIN DISTRIBUTIONS AT ℄ FOR BEAMS OF LOAD GROUP #3

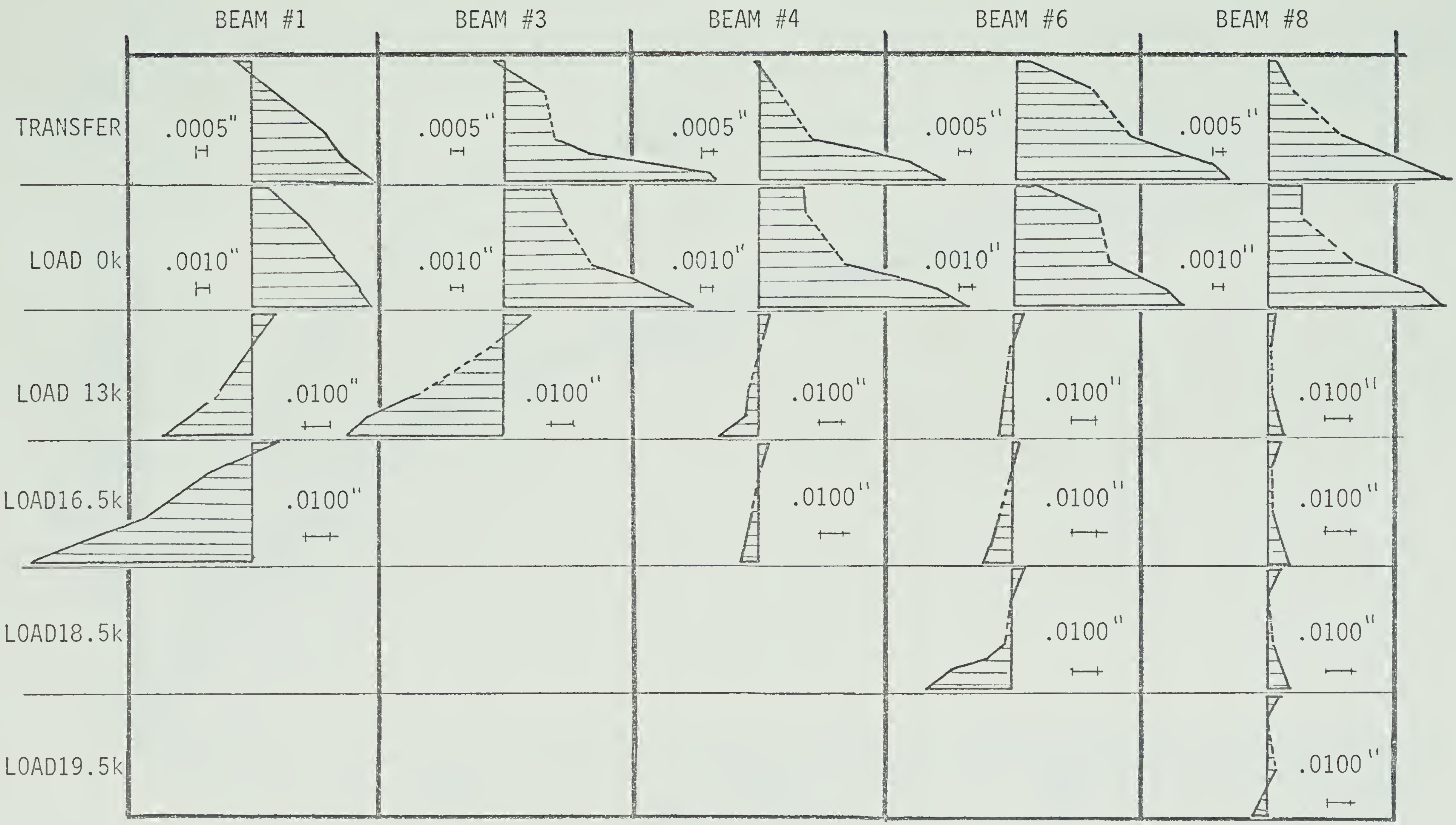


FIGURE 4.2(b) STRAIN DISTRIBUTIONS AT ℄ FOR BEAMS #1 AND OF LOAD GROUPS 2 & 4

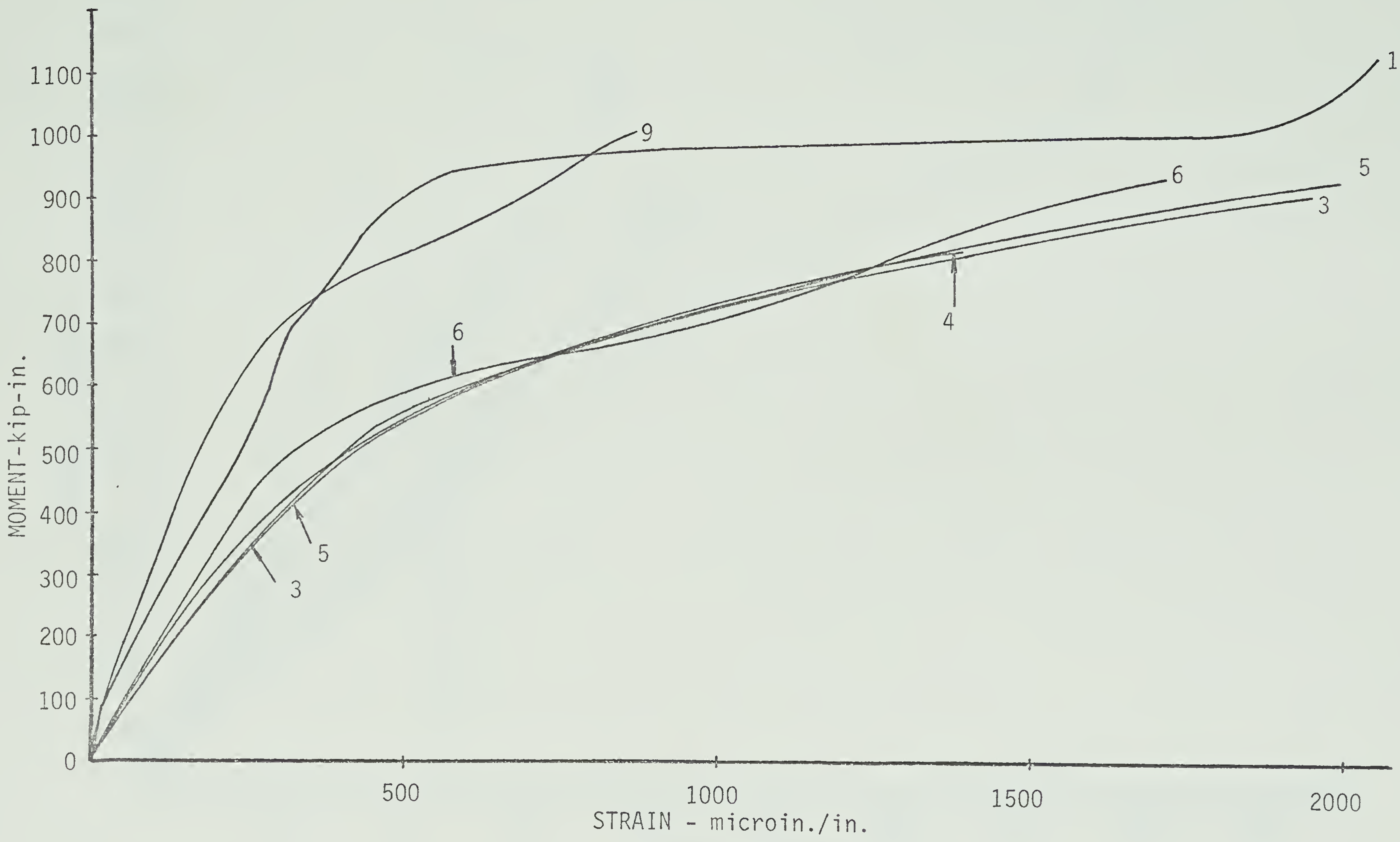


FIGURE 4.3 (a) MOMENT-STRAIN RELATIONSHIP FOR BOTTOM LONGI. REINF. UNDER LOAD POINT

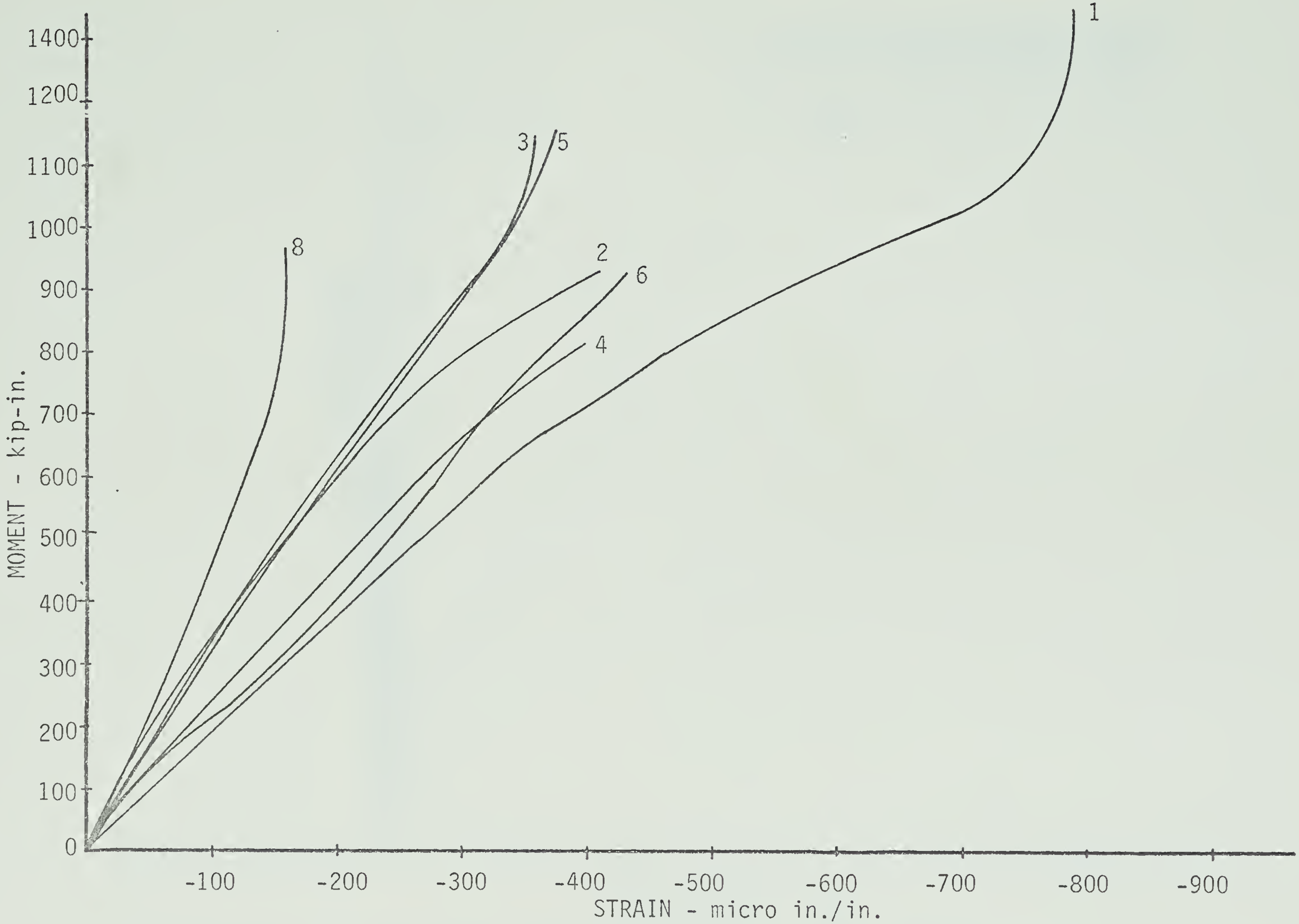


FIGURE 4.3 (b) MOMENT-STRAIN RELATIONSHIP FOR TOP LONGI. REINF. AT CENTERLINE

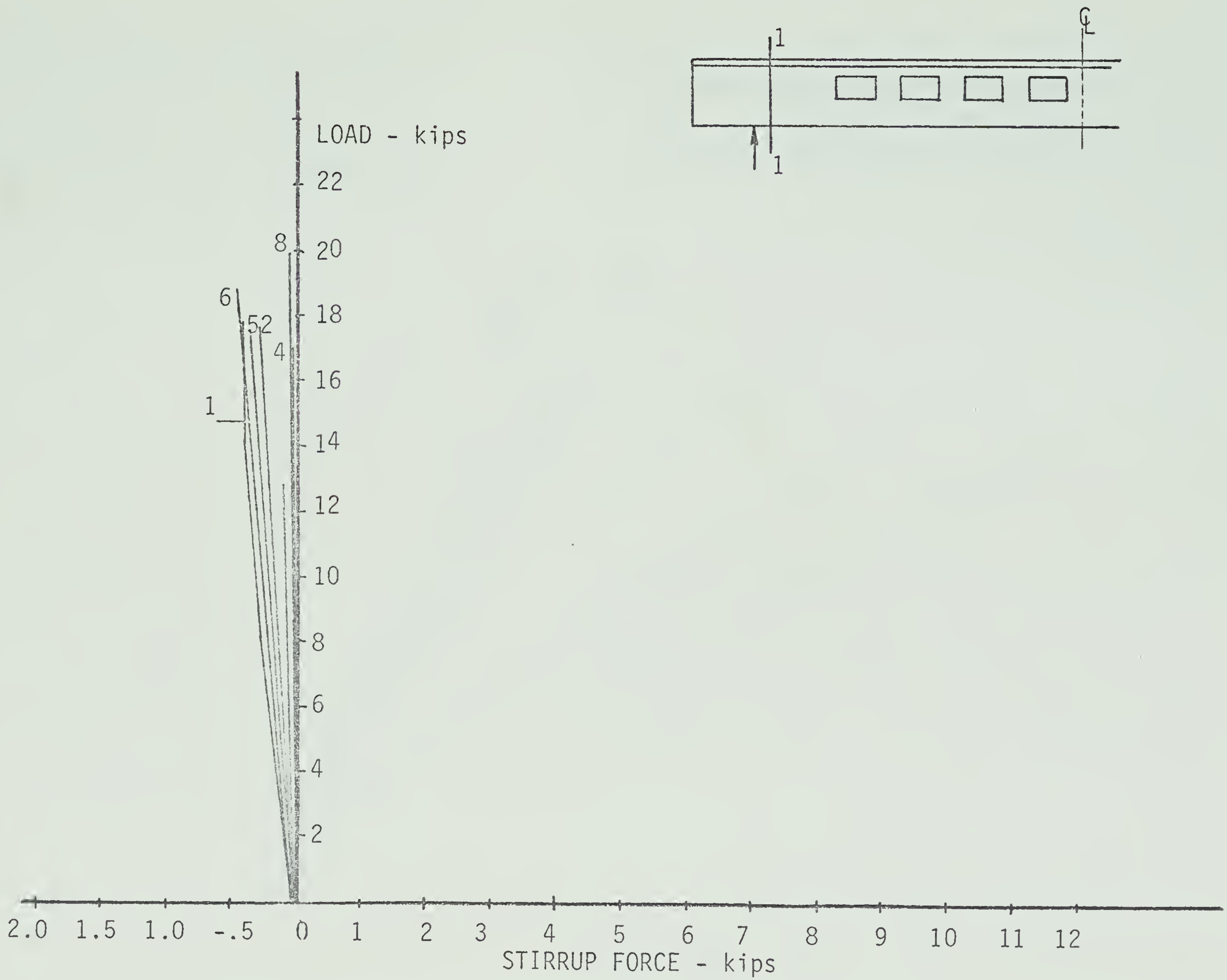


FIGURE 4.4 (a) LOAD-STIRRUP FORCE RELATIONSHIP AT SHEAR SECTION 1

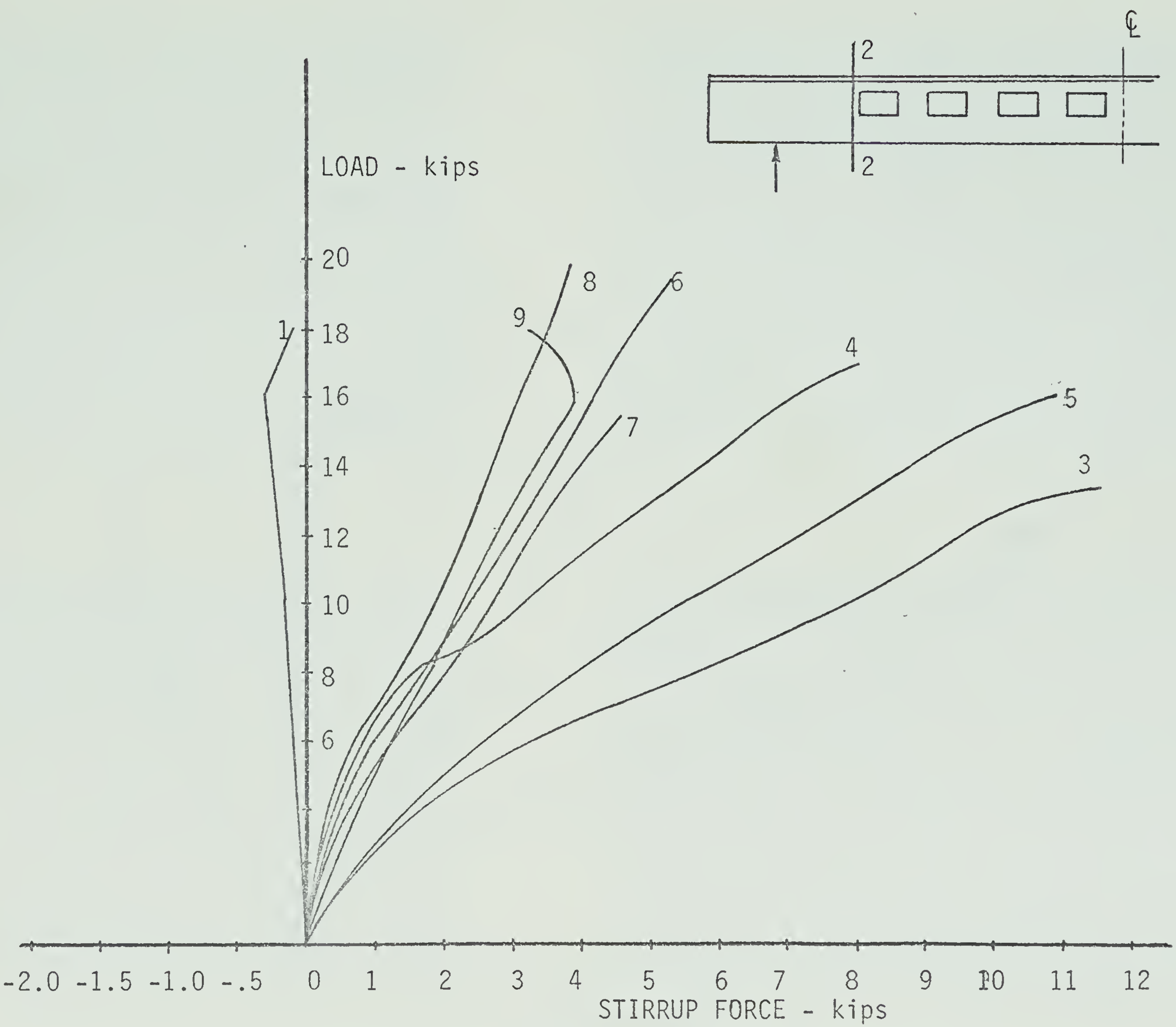


FIGURE 4.4 (b) LOAD-STIRRUP FORCE RELATIONSHIP AT SHEAR SECTION 2

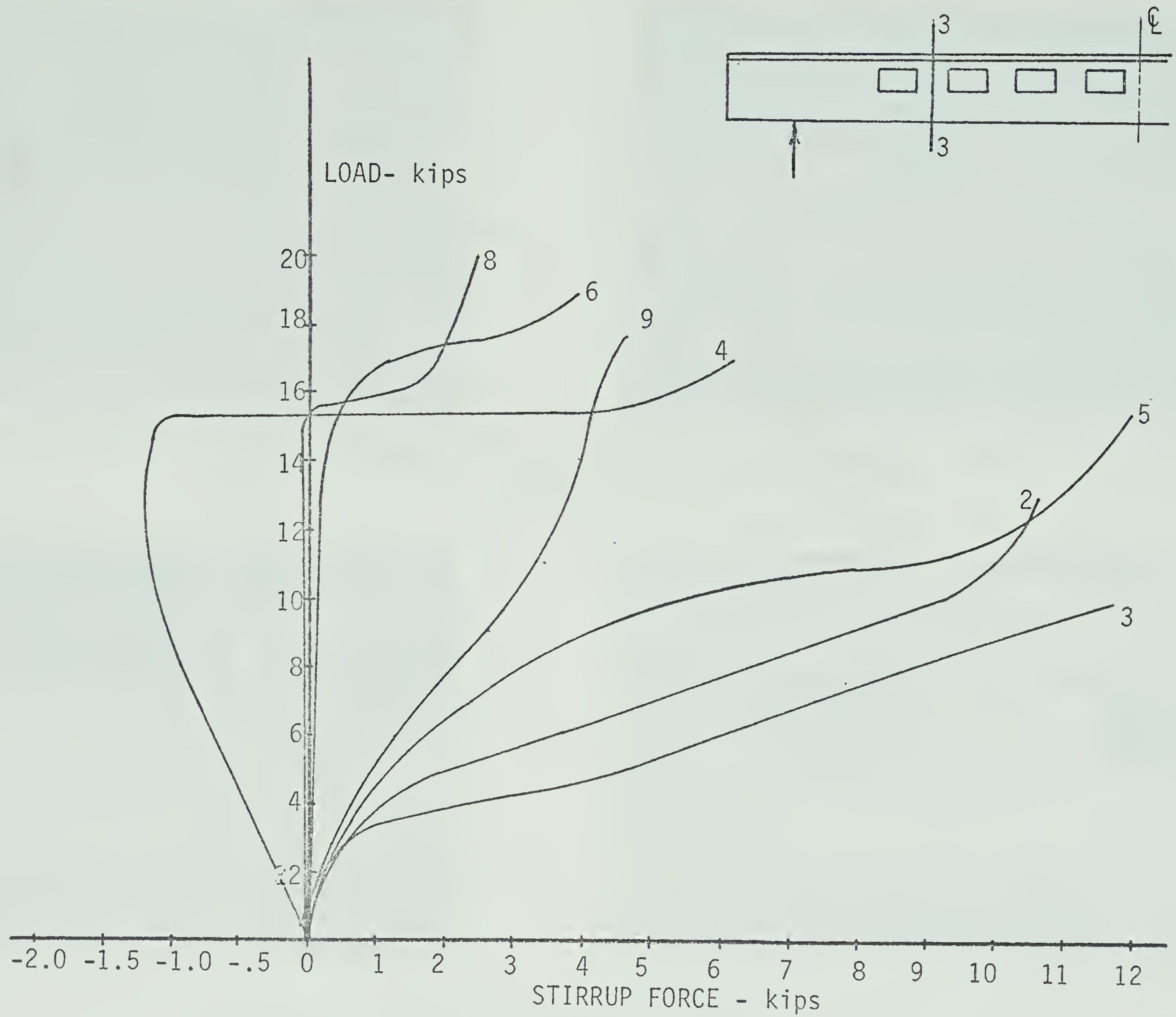
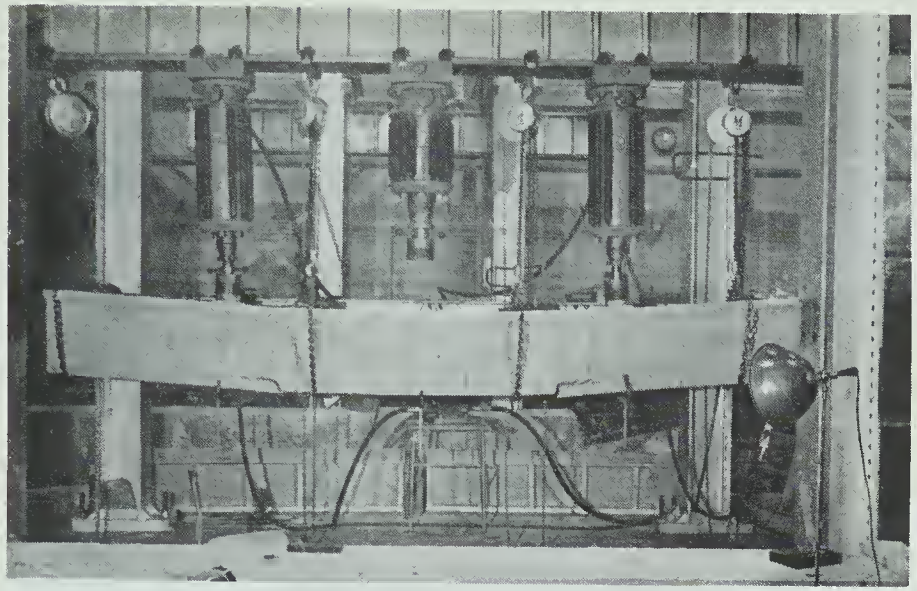


FIGURE 4.4 (c) LOAD-STIRRUP FORCE RELATIONSHIP AT SHEAR SECTION 3



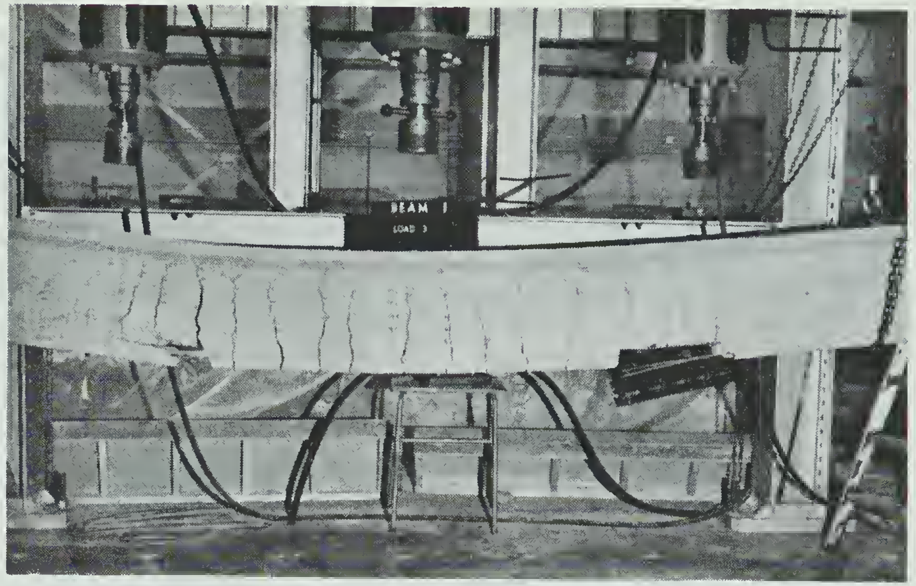
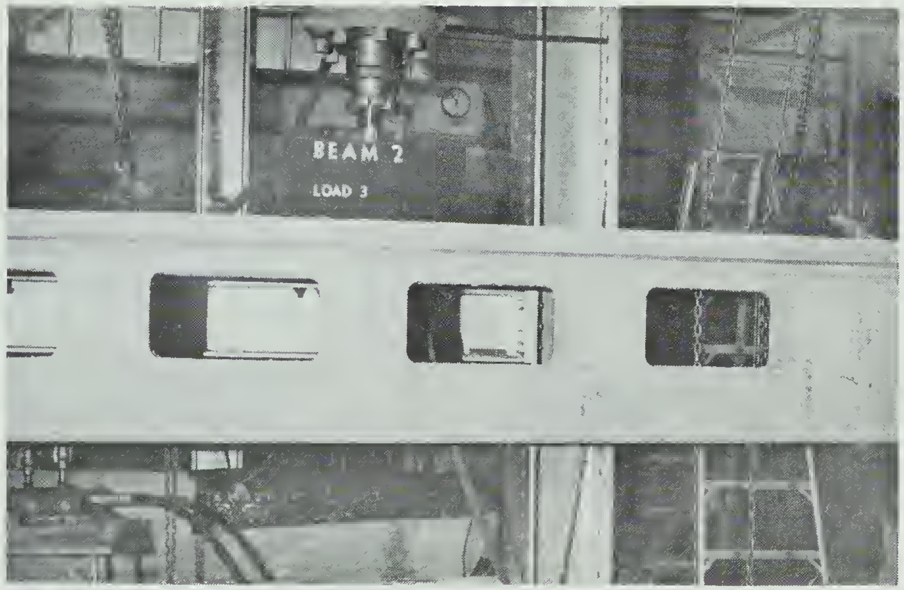


FIGURE 4.5 BEAM #1 CRACKING AND FAILURE PATTERNS



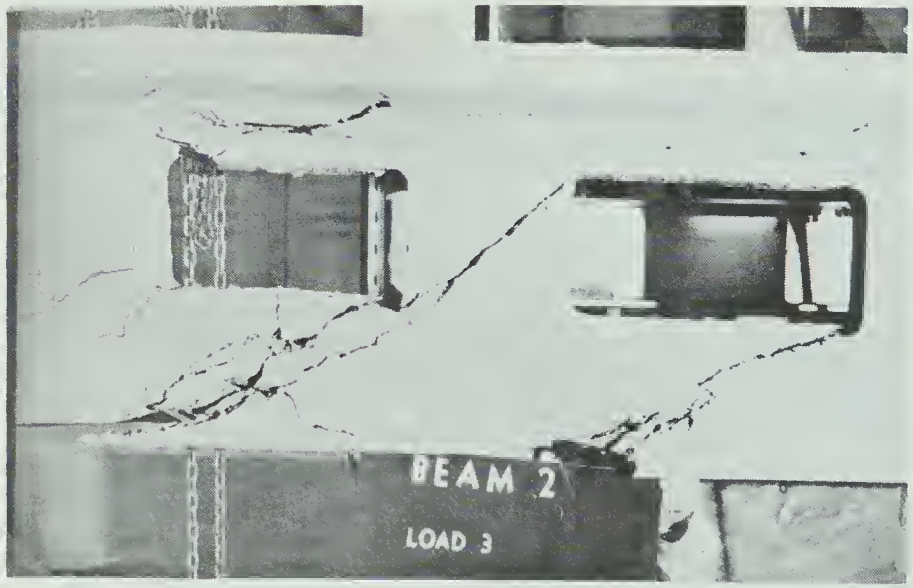
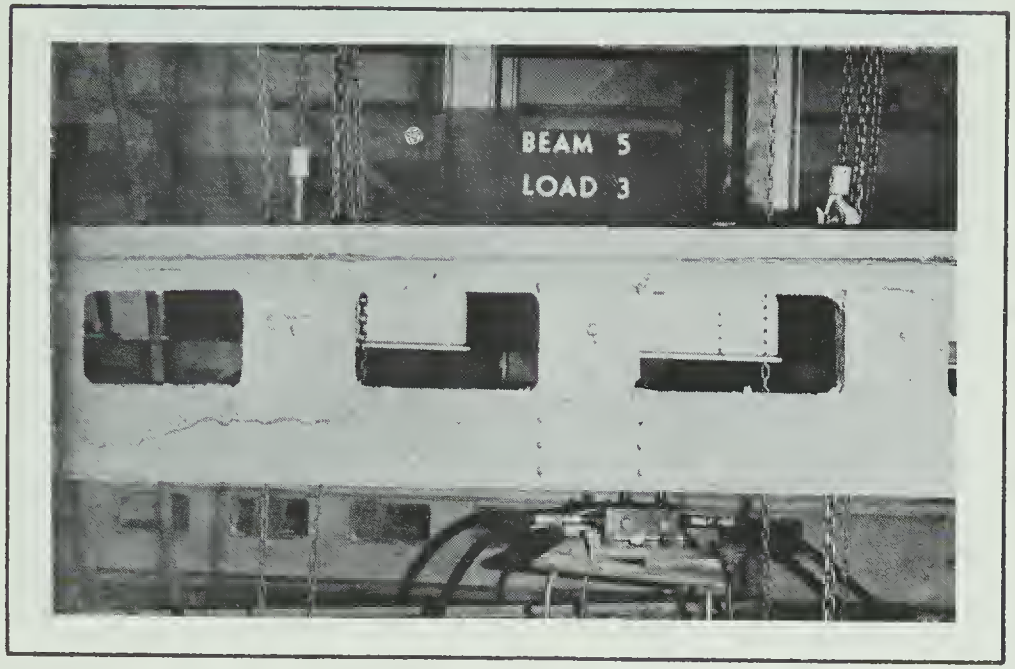


FIGURE 4.6 BEAM #2 CRACKING AND FAILURE PATTERNS



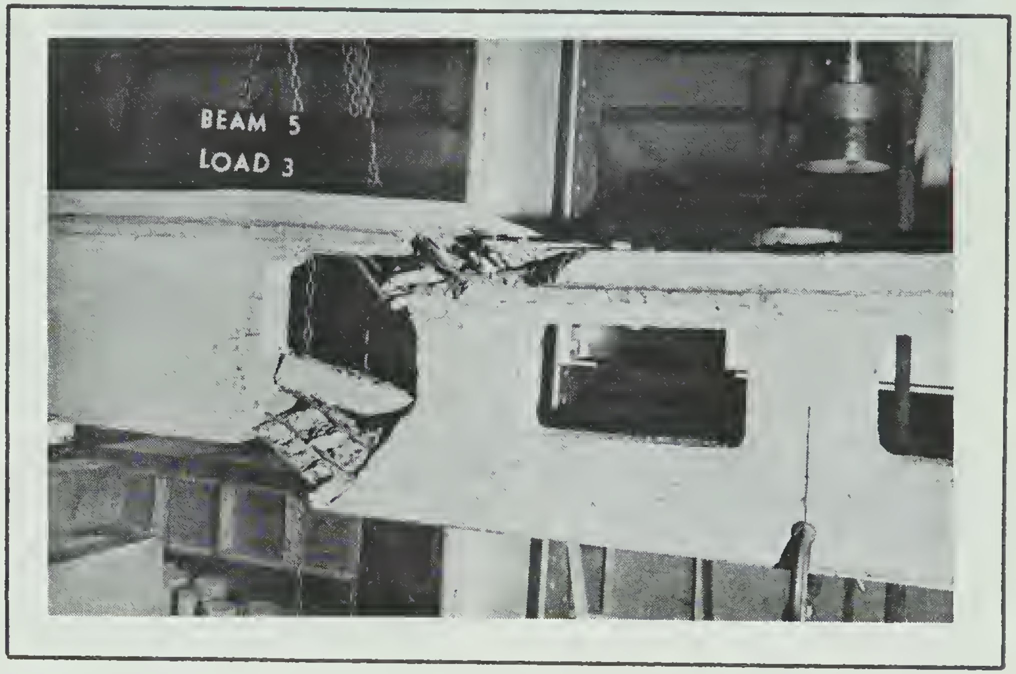
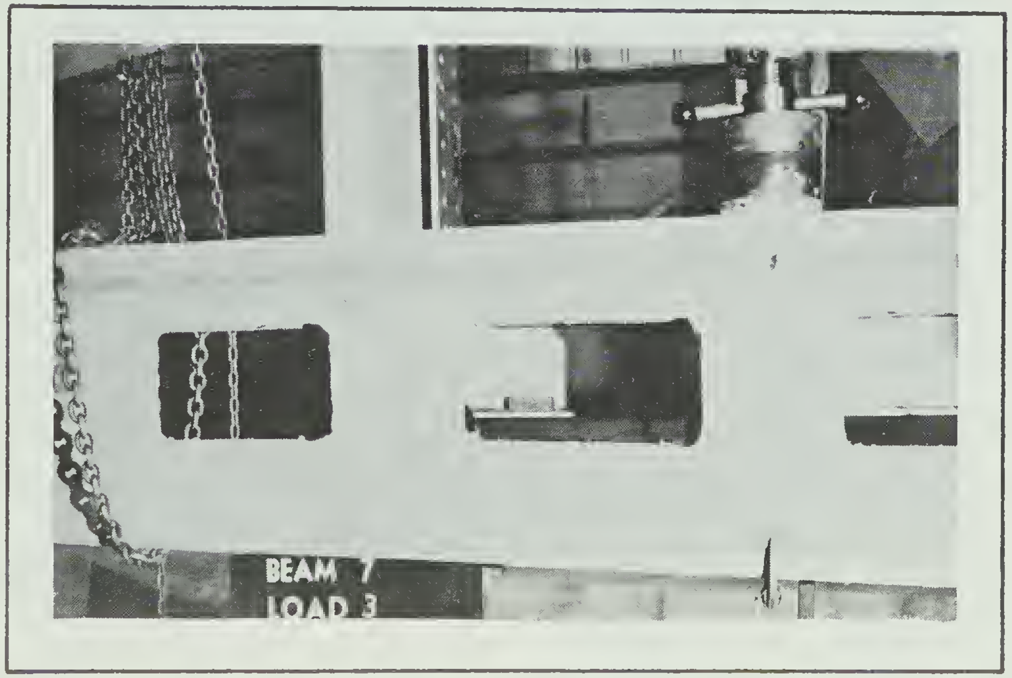


FIGURE 4.7 BEAM #5 CRACKING AND FAILURE PATTERNS



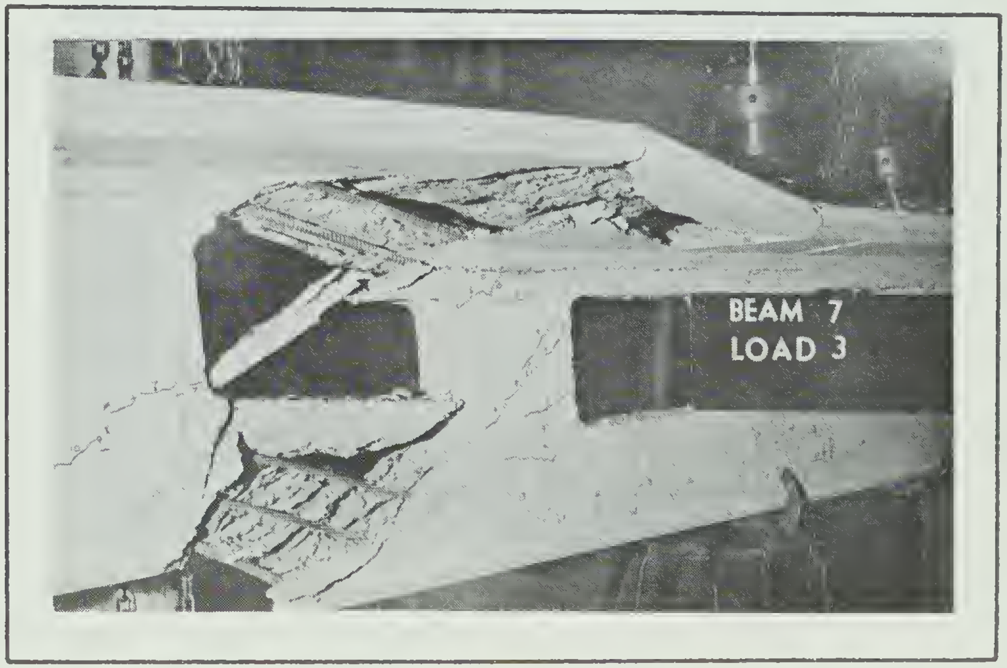


FIGURE 4.8 BEAM #7 CRACKING AND FAILURE PATTERNS

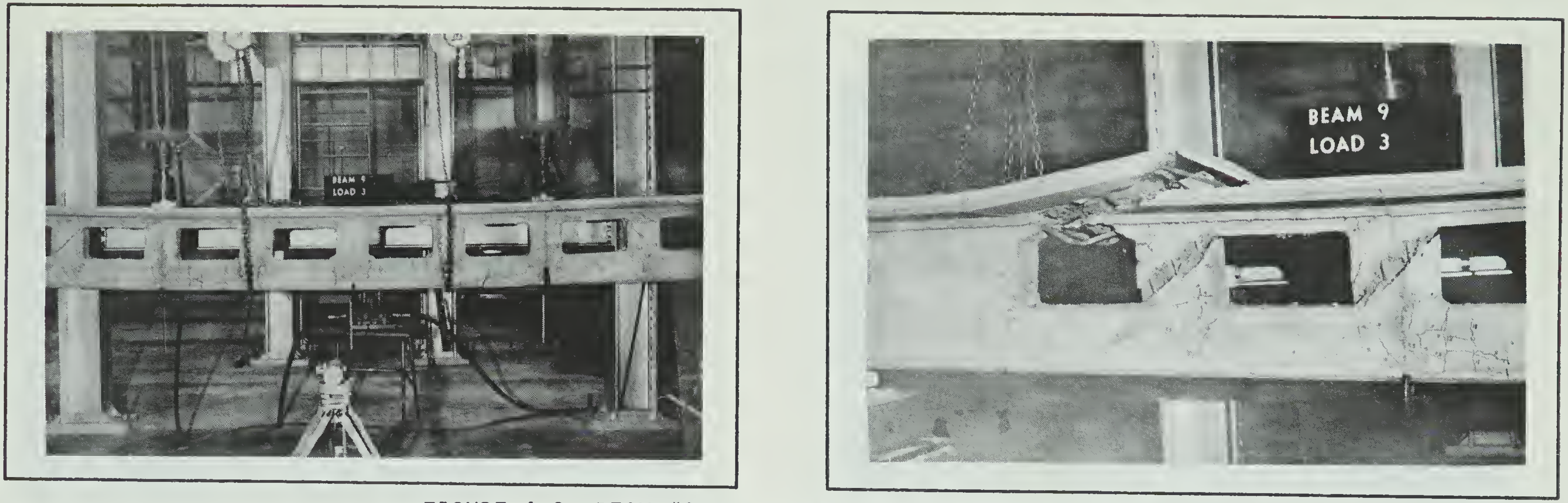


FIGURE 4.9 BEAM #9 CRACKING AND FAILURE PATTERNS

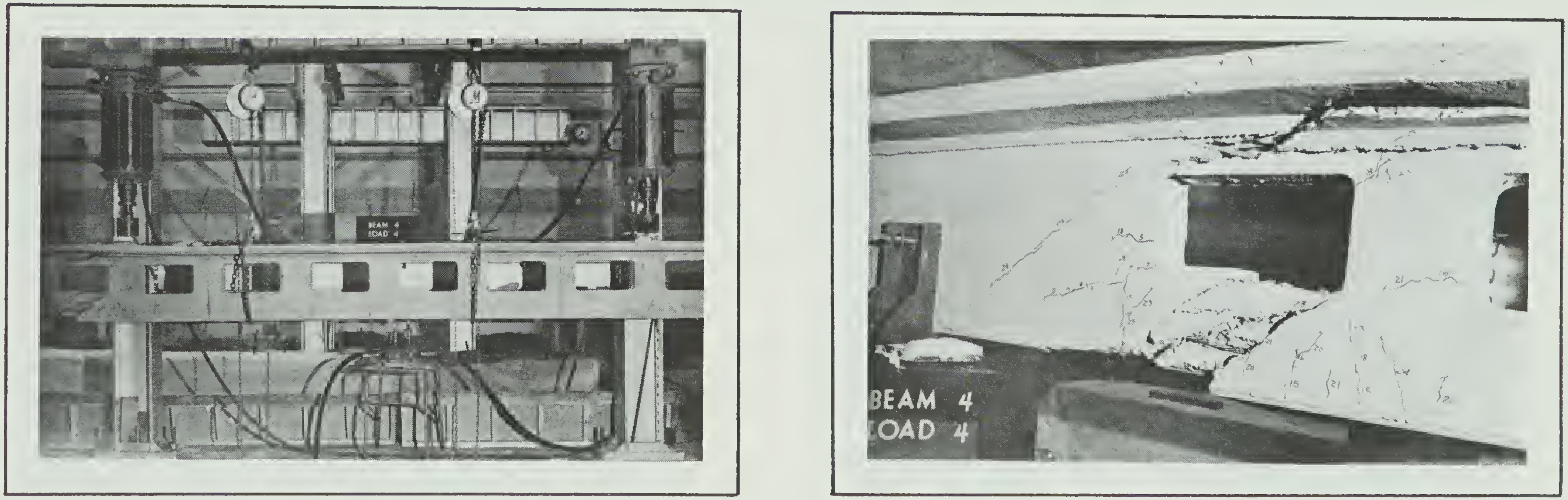


FIGURE 4.10 BEAM #4 CRACKING AND FAILURE PATTERNS

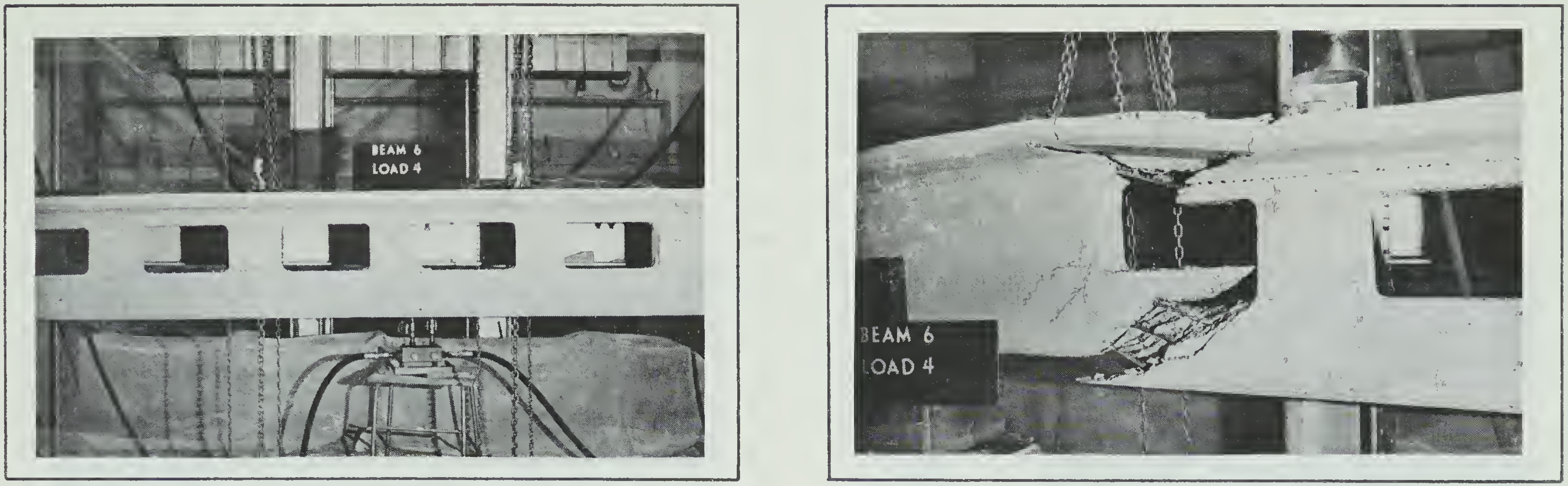


FIGURE 4.11 BEAM #6 CRACKING AND FAILURE PATTERNS

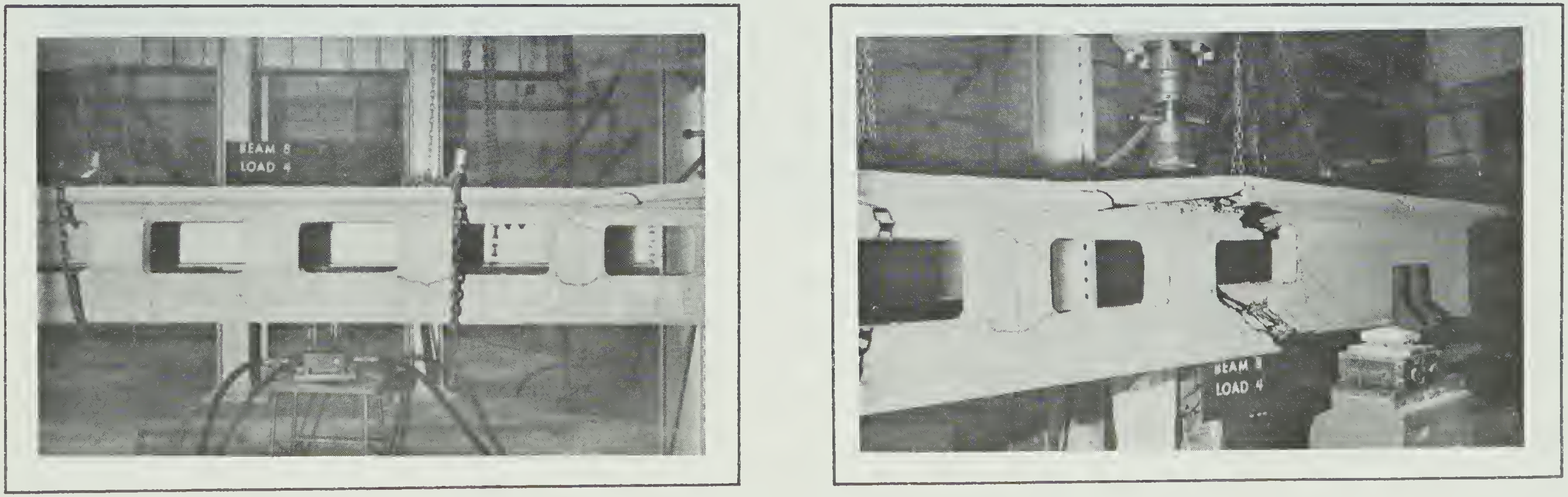


FIGURE 4.12 BEAM #8 CRACKING AND FAILURE PATTERNS

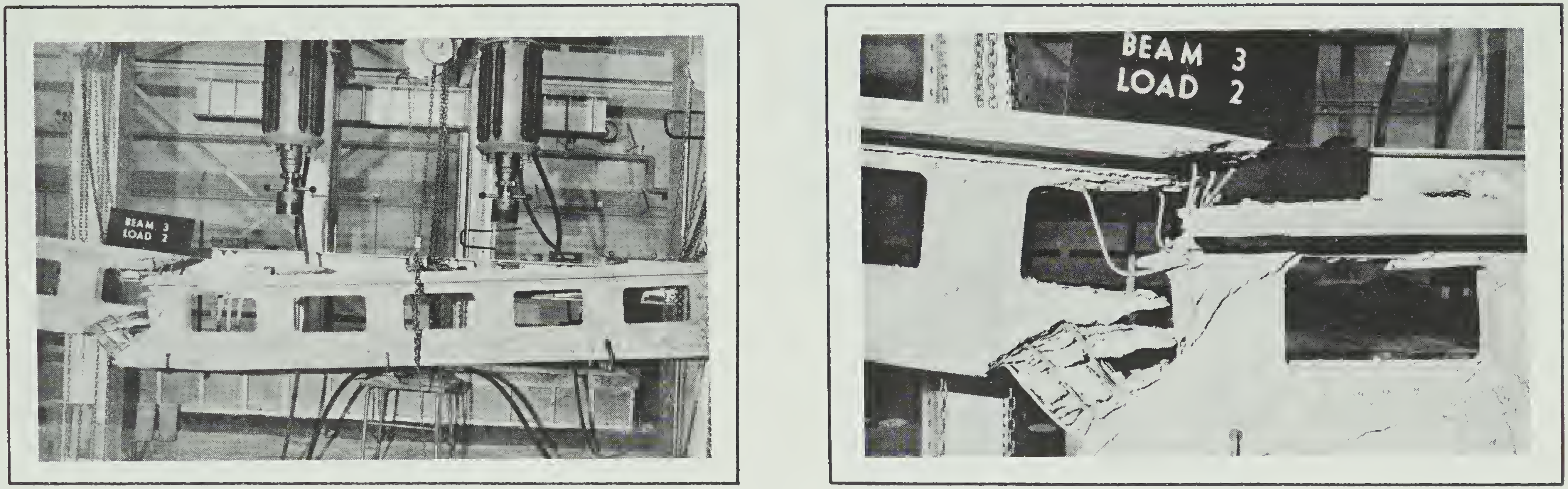


FIGURE 4.13 BEAM #3 CRACKING AND FAILURE PATTERNS

CHAPTER V

DISCUSSION

In this research program, nine beams were tested, eight of which contained large rectangular web openings. Since each beam had three differing parameters, no two members behaved identically. This chapter presents a discussion of the parameters, behaviour of the beams and of the test results obtained. One of the parameters used was varying load positions and for convenience in the following discussions and comparisons, the beams have been grouped according to their loading position.

5.1 Control beam

Beam #1 was fabricated for purposes of comparison and contained no holes. Its ultimate design capacity was 940 k-in. according to the ACI code[4], but its actual ultimate moment was 1460 k-in. . The only explanation apparent to the author for this discrepancy is the use of the safety factors involved in the ultimate tensile strength of the prestressing strands, f_{su}. Using f'_s, that is f'_s= 250 ksi instead of f_{su}= 188 ksi, the flexural failure load would be 17.5 kips per jack instead of 13.5 kips. It is to be noted that this beam was designed as an underreinforced member to fail in flexure and not in shear. Design calculations are presented in Appendix C.

Throughout testing, this beam behaved in a ductile manner

and from the cracking patterns, as shown in Fig. 4.5 and the load-deflection curves, it is very reasonable to say that this beam behaved as a typical underreinforced prestressed Tee beam. Flexural cracks were first observed at a load of 9.5 kips per jack and progressed rapidly to failure. Since load was applied by two symmetrically placed jacks, the flexural cracks occurred at regular intervals within the pure moment span. The cracks opened considerably especially at sections located approximately one foot in from the load points. At these sections, a concentration of stress occurred as the supplementary longitudinal reinforcement terminated here. The reason for having this projection of reinforcement into the pure moment span was to develop sufficient bond in this reinforcement to enable measurement of the strain by strain gages located under the point of load.

Shear reinforcement in this beam consisted of #3 stirrups spaced at 12 in.. This spacing was based on the minimum stirrup spacing required by the ACI code[4] 1963 and 1971. As seen from Figs. 4.4 (a) and (b), the stirrups at shear sections 1 and 2 were of little use as it appears from the strain readings that these were in compression. The effect of the stirrups at shear section 3 was unknown as the strain gage at this section was inoperative. The general pattern indicates however, that shear reinforcement had little if any effect on the shear capacity of the beam.

The strain distribution diagrams at centerline in Fig. 4.2 (a) and (b) also indicate a typical ductile behaviour with the

neutral axis high in the flange near failure. The estimated location for the neutral axis at failure was 0.7 in. from the top fiber and the flexural cracks at failure penetrated the flange to a distance of approximately 0.6 in. from the top fiber.

Failure of this beam resulted from the visible fracture of one of the prestressing strands. This strand snapped at approximately one foot in from the load point, that is, at the point of concentration of stress at the projected end of the supplementary longitudinal reinforcement. The total vertical deflection at centerline at failure was in excess of 11 in. and after unloading, the beam was permanently deflected by approximately 7 in. at the centerline.

5.2 Discussion of the beam parameters

The beam parameters used in this investigation were those of varying load positions, vertical shear reinforcement and supplementary longitudinal reinforcement. For each load position, beams were constructed having different shear and longitudinal reinforcements, as shown in Fig. 3.4.

Varying the load position had the effect of changing the applied shear to maximum moment conditions; that is, load position 4 resulted in a more severe shear to moment condition than load 2 which resulted in a less severe condition relative to moment. Load position 3, however resulted in an average shear to flexure condition which might best represent the moment condition found in most structures, which would result from a uniformly distributed load.

However, the shear condition produced by load position 3 is much more severe than that corresponding to a uniformly distributed load.

As stated previously, each beam in each group had differing amounts of shear and/or longitudinal reinforcement. This permitted a direct comparison of the beams within each group as the other parameters were varied; consequently, the effects of changing the vertical and longitudinal steel were studied and interpreted directly. The addition of an extra stirrup in the posts subjected to high shear and low moment, that is beams of group 4, resulted in a significant increase in shear capacity, while an even higher increase resulted from the extra stirrup in beams of group 3. Since group 2 consisted of beam #3 alone, only general relationships could be established between it and the other groups. The use of #4 longitudinal bars , instead of #3 bars, had very little effect on the shear capacity of the members; in one case, the shear capacity was slightly decreased in a beam containing #4 bars. The anticipated effect of having a larger longitudinal steel area was to confine the openings, subjected to shear and moment, by resisting the horizontal component of diagonal tensile stress by tensioning the bars and resisting the vertical component of diagonal tensile stress by doweling resistance.

5.3 Load group 3

Load group 3 was made up of beams #1,2,5,7 and 9. A summary of the beams' reinforcements and test results are presented in

Chapters III and IV respectively. It was previously noted in section 5.1 that beam #1, the control beam, behaved in a ductile manner, as designed. Its use was practical in the sense that it served as a basis for relating results obtained from the other beams. Had this beam been loaded at positions 2 and 4, anticipated flexural failure would have occurred at jack loads of 15.2 and 30.5 kips respectively.

Beam #2 was a direct equivalent of the control beam as the parameters used were identical; the only difference is that beam #2 had large openings, hence the stirrups had to be concentrated in the posts. This "bunching" up of the reinforcement together with the presence of the openings led to high diagonal stress concentrations in the concrete in the lower webs and flanges of the first two holes and in the post adjacent to these two openings. Consequently, the beam failed in shear through severe cracking of the lower webs, flanges and post at a load of 13.5 kips per jack. As indicated from the load-deflection curve, the strain distribution diagrams and the moment-strain diagram, beam #2 behaved similarly to beam #1 up to the failure load; the only difference in this range was that the total average deflections of beam #2 were close to being twice those of the control beam. This verifies Nassers'[3] observations. It should be noted at this time that the load which caused shear failure in this beam has the same value as that predicted by the ACI code(1963)[4] to cause flexural failure.

The next beam in this group, beam #5, contained 3#3 stir-

rups per post, being equivalent to having the stirrups spaced at 8 in. in a beam containing no holes. The beam's load-deflection curve in Fig. 4.1 (a) shows that it sustained smaller deflections than beam #2 for the same load. Strain distribution diagrams at centerline show that, at the extreme fibers, the strains are approximately half of those of the control beam. The linearity of strains within the web of beam #1 however, is not attained in beam #5. The presence of the openings caused remarkable strain disruptions over the depth of the section. This phenomenon was also observed when the beam was undergoing cracking; some of the flexural cracks in the centerspan, between jack loads of 11.5 and 16.0 kips, were observed to commence at the top fiber of the lower web and to work down toward the bottom fibers of the beam. These cracks were a direct result of the flexural stress concentrations at the edges of the openings.

For beam #5, the addition of the extra stirrup in the posts contributed significantly to an increase in its shear capacity which was greater, by 3.0 kips, than that for beam #2. Furthermore, the extra stirrups caused failure to be confined through the lower web and flange of only one hole in contrast to the failure of beam #2 where two holes were affected. This is illustrated in the photographic plates of Chapter IV, in Figs. 4.6 and 4.7. The extra stirrup was also successful in resisting the diagonal tensile forces in the posts, as failure cracks were not present in these posts.

Further phenomenal behaviour was observed at failure. The posts in the pure moment span were sheared at the top and bottom of the openings. Since these posts contained no reinforcement, it is to be expected that they had little transverse shear resistance. The transverse shear resulted most likely from a sudden release of energy . This release occurred as a result of shear failure through the web and flange which caused the energy stored in the prestressing cables to be transfered when the cables became slack at the location of failure, thus causing differential shear between the flange and the lower web.

Besides having three stirrups per post, beam #7 also contained #4 supplementary reinforcement above and below the openings in the shear spans. Although it was expected that this beam would likely behave in a stiffer manner than beam #5, due to large increase in longitudinal steel, the load-deflection relationships show otherwise. At loads between 0 and 12 kips, beam #7 sustained larger deflections for the same load. At loads above 12 kips however, it behaved identically to #5, except that finally beam#7 had slightly lower shear capacity; shear capacity was 1.0 kip lower than that of beam #5. Since it was anticipated that beam #7 would have had a larger shear capacity, due to the presence of the #4 bars, then the reduced capacity should be attributed to fabrication flaws.

From the strain distribution diagrams, shown in Fig. 4.2 (a), beam #7 had larger compressive strains in the bottom fibers at "0" load; this would then explain the higher load required to

cause the first flexural cracking. The larger compressive strains obtained could possibly have been a direct result of the shorter moist curing period and also of the earlier time of transfer of the prestress.

In the plots of Fig. 4.4 (b), there was a general trend that the stirrups of beam #7 were subjected to a lesser force than those of beam #5 for the same applied load. This could be a result of the additional longitudinal steel assuming a proper functioning of the electrical strain gages.

Beam #9 was reinforced similarly to #7, except that it was also provided with 2#3 inclined U-stirrups in the two lower webs in each shear span. From Fig. 4.1 (a) it can be observed that the beam's load-deflection behaviour was very similar to that of the control beam. Beam #9, when compared to beams #5 and 7, was observed to exhibit smaller deflections at any given load up to approximately 16.5 kips.

The strain distribution diagrams for beam #9 show that the strain progression at centerline was very much like that of the other beams in this group; the usual strain disruptions near the edges of the openings were apparent also from the cracking as some of the flexural cracking was initiated just below the openings and worked down toward the bottom fibers.

After an applied load of 11.0 kips per jack was reached, shear cracking almost stopped progressing but flexural cracking continued at a fairly steady rate. A load of 17.5 kips was main-

tained for approximately three minutes before failure occurred. Failure was the result of the shear of the two posts located in the north shear span and was quite sudden. At the same time, the center posts were sheared off at the top and bottom, as in beam #5. A large longitudinal crack was also observed at the level of the prestressed reinforcement, at centerline, and this crack spanned approximately four feet. Since complete collapse had not occurred, the beam was subjected to slightly more deflection which caused buckling up of the flange above the first opening. No major shear cracking was observed in the lower webs.

Although the deflection behaviour of this beam was not unlike that of the other beams, the failure behaviour was remarkably different. In beam #2, shear failure occurred through the lower webs, flanges and posts, in #5 and 7 failure occurred through the lower web only, due to the extra "post"stirrups provided and in beam #9 failure occurred through the flange and two shear posts only. From the electrical resistance strain gage readings presented in Appendix C, it is quite evident that the extra U-stirrups provided, served to redistribute the combined shear and flexural stresses; the diagonal tensile stresses were distributed more evenly in the shear span, that is, stress concentrations were not as great due to the "linking" effect provided by these stirrups.

5.4 Load group 4

Load group 4 was made up of beams #4,6 and 8. These beams were subjected to a large shear and small moment, relative to beams of group 3 and 2.

Beam #4 was the first in this series, and was reinforced with two stirrups per post or equivalent spacing at 12 in., and #3 supplementary longitudinal reinforcement, as shown in Fig. 3.4. Being subjected to large shear and small moment, the beam was stiffer than those of load group 3, as shown in Fig.4.1 (b). Centerline strain distribution linearity is quite apparent for this beam in Fig. 4.2 (b); the strains are linear up to the penultimate load. This was to be expected as the flexural stresses caused by this load of 16.5 kips at this load position were much smaller than those caused by a 13.0 kip load at position 3. Further ultimate behaviour of this beam exemplified Lorentsen's[2] theory that holes in beams should be placed away from shear regions. This beam had a minimum amount of web opening in the shear span, as only one hole was located beyond the pure moment span; consequently, higher shear capacity was attained. It is then to be expected that if no web openings were present in the shear spans, in the beams of group 3 and 4, then flexural and not shear would be the ultimate failure critireon. Figures 4.4 (b) and (c) illustrate this in a trend that indicates that the stirrup force at shear sections 2 and 3 lessens by approximately 2.5 kips or more for each hole removed from the shear span subjected to the same load.

The next beam in this group, beam #6, had three stirrups per post or equivalent 8 in. spacing, and #3 longitudinal reinforcement, as depicted in Fig. 3.4. This beam, also being subjected to large shear and small moment, behaved similar to beam #4 except that it was slightly less stiff due to the closer spacing of the stirrups which resulted in supplying a slightly higher shear capacity, hence

larger deflections at failure; beam #6 had an increase in shear capacity of 2.5 kips. As in beam #4, strain distribution linearity at centerline was quite apparent up to a jack load of 16.5 kips as shown in Fig. 4.2 (b). At a load of 17.5 kips, it was observed, as in beam #5 at a load of 11.5 kips, that some flexural cracks in the lower web started at the bottom of the opening and worked down toward the bottom most fibers. The moments required in beams #5 and 6 to cause these strain concentrations were 830 k-in. and 840 k-in.respectively. Consequently, it would then be expected that the strain distribution at centerline would lose its linearity; this in effect occurred as shown in Fig. 4.2 (b) at a load of 18.5 kips. A similar behaviour occurred in this beam that also occurred in beam #5; the posts in the moment span were sheared off at the top and bottom from the upper and lower web respectively, due to the redistribution of prestress at failure, that is at a load of 19.5 kips in this case.

Beam #8 is the last in this series under load position 4 and did not behave much differently than beam #6. The load-deflection diagram for beam #8 indicated a very slight difference in behaviour compared to that of beam #6. Beam #8 did however have a slightly higher shear capacity; 0.5 kip higher. This extra capacity could be attributed to the extra supplementary reinforcement used; #4 bars as compared to #3 bars in beam #6. This gain in strength is so small however, that it could also have been caused by material quality, or by small variations in the placing of the steel.

The strain distributions obtained for beam #8 were somewhat

puzzling as these were quite different during loading than those of any other beam. Only at the penultimate load were the bottom fibers in tension, according to these distributions; this however was not the case as observed from the cracking patterns, as flexural cracks were recorded at loads of 14 kips and higher. Hence, the only possible explanation for this behaviour would be the localizing of strains at the centerline. This would occur if flexural cracks were formed on each side of the Demec points, thus causing this center block to behave somewhat independently from the rest of the beam. Upon close examination, cracks were observed to be present on each side of the lines of Demec points.

5.5 Load group 2

Load group 2 consisted of only beam #3. This beam was subjected to small shear and large moment. It behaved in a very ductile manner, as was expected, since every increment of load produced a moment 1.33 times that produced by an increment in group 3. Each increment of load also produced deflections which had values averaging eight times those produced by any increment in group 4. The strain distributions at centerline also indicate this type of behaviour. Although this beam had a low shear failure load, 13.9 kips, it sustained a large bending moment. It is believed that, had the stirrups been placed three per post instead of two per post, then flexural failure would have occurred; the extra stirrups would have increased the shear capacity by approximately 3.0 kips, as in beam #5, hence causing a moment of magnitude greater than that in beam #1. This beam

served to verify Lorentsen's[2] theory that the addition of openings in the shear span reduces the beam's load carrying capacity, as explained in the discussion of beam #4 group 4.

5.6 General discussion

All the beams in this experimental project failed by shear except for beam #1 which failed by flexure. General beam behaviour varied from very ductile to very brittle; the type of behaviour was directly affected by the loading position and less directly by the reinforcement used. Beam #1 of group 3 and beam #3 of group 2 are those which behaved most like members desired for a structure because of their ductile behaviour, as depicted by their load-deflection diagrams. Although beam #3 behaved in an ideal fashion, its set-up was not practical as it was subjected to large moment but small shear. On the other hand, beams of load group 4 were in the other extreme position as they produced very brittle behaviours due to small applied moments but large shear loads. Hence the most practical behaviour patterns would be associated with the beams of load group 3 as these were subjected to both large shear and large moments.

The photographic plates of the cracking and failure results illustrated in ChapterIV , being grouped according to their load position show the different patterns resulting from varying the vertical and supplementary longitudinal reinforcements.

The effects of increasing the vertical or shear reinforcement resulted primarily in increasing the shear capacity of the

members; by 15 % to 22 % in load group 3 and by 18 % in group 4. A second effect of the increase in vertical steel area was to confine the shear failure to one opening only, as seen in beams #5,6,7 and 8. Beside confining the failure, reinforcing the posts also gave these posts the extra capacity required to resist the diagonal tensile stresses except in beam #9, where the addition of inclined U-stirrups in the lower webs of the shear spans resulted in a redistribution of stresses causing failure to occur in these posts. Extra inclined U-stirrups however, boosted the shear capacity of the beam by 18 %.

The increase in supplementary longitudinal reinforcement did not cause as drastic improvements in beam behaviour as did the addition of vertical steel. In beam #8 an increase in shear capacity of only 3.8 % was obtained while in beam #7, a decrease in capacity occurred; this decrease is not substantiated however, by any other tests and possibly a flaw in the beam could have caused this decrease. The overall effect then of increasing the steel area in the posts and in the lower webs resulted in a more complete and efficient beam behaviour.

CHAPTER VI

SUMMARY, CONCLUSIONS, RECOMMENDATIONS

6.1 Summary

Nine simply supported 24 foot prestressed concrete Tee beams containing large web openings were tested under two varying point loads. The beams were grouped according to their loading positions, that is, group 2, group 3 and group 4. Other variables included vertical or shear reinforcement and supplementary longitudinal reinforcement. The beams' behaviours were recorded in terms of applied loads and resulting deformations. Of particular interest were the changes in beam behaviour and the effect of varying the vertical and longitudinal reinforcement on the load carrying capacity under different loading conditions. This series also indirectly attempted to expose any difficulties in the procedures and methods used, which could be improved upon for future tests in the program.

6.2 Conclusions

The following conclusions are based on the results of tests conducted on nine beams, each of which contained three previously mentioned independent variables.

The conclusions are summarized as follows:

a) Prestressed concrete Tee beams containing large web openings can not be designed for flexure using the ACI code's minimum shear reinforcement conditions even though the actual ultimate shear capa-

city obtained could far exceed that designed for flexure.

b) Any additional shear reinforcement provided, served to increase the load carrying capacity of a beam containing large web openings by an amount ranging from 15 % to 22 % .

c) Additional shear reinforcement also confined failure to a location where there was an abrupt change in cross - sectional area of concrete.

d) An extra amount of vertical reinforcement placed in the posts gave these posts the capacity required to cause a localizing of the failure in the lower web if this web had no vertical reinforcement. However, a minimum amount of inclined shear reinforcement placed in the lower webs caused the failure to be localized in the posts.

e) The addition of both post and lower web reinforcement resulted in a redistribution of stresses in the shear span such that all sections were more equally stressed in diagonal tension.

f) A considerable increase in supplementary longitudinal reinforcement did not significantly increase the shear capacity of the beams.

g) A decrease in the number of openings in the shear span increased considerably the shear capacity of the beams.

6.3 Recommendations

a) The present test series was not sufficiently complete, as no flexural failure was obtained in the beams containing the openings.

Hence further testing should be supplemented such that the variables used be more suitably changed to cause complete flexural behaviour in beams containing openings.

b) It is recommended that further testing concentrate on studying the effect of vertical reinforcement and not supplementary longitudinal reinforcement as the former proved more efficient in increasing the shear capacity of the members.

c) Other variables which should be more extensively studied are the number and size of openings provided in the shear spans.

d) Further studies on such members should also include effects of increased longitudinal prestress reinforcement, so that some failures would occur by crushing of the concrete, as all the beams in this program were under reinforced and no concrete crushing took place.

e) It is suggested that the procedures, methods and equipment used in this present program be adopted for future tests as all stages of preparing, casting and testing proved to be very effective and extremely simple.

f) Finally it is recommended that the type of formwork used also be adopted as placing and removing of the forms was rapid and accurate.

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APPENDIX A

MATERIALS AND PROCEDURES

APPENDIX A

A.1 Materials

a) Cement

Type III, high-early strength, Portland cement was in all mixes.

b) Aggregate

The sand had a fineness modulus of 2.53 and an average moisture content of approximately 4%. The coarse aggregate used was a pea-gravel with a maximum size of 3/8". Sieve analysis for the sand and coarse aggregate are presented in Tables A.1 and A.2. Both aggregates have been used in this laboratory for several previous investigations and have passed the usual specification tests.

c) Concrete mix

A satisfactory mix has been developed by Wilkinson[5] using the same materials. The same mix design was adopted for this present test series. The ratios, by weight, per batch were:

cement	1.0	(170lb)
sand	2.2	(380lb)
coarse aggregate	1.6	(270lb)
water	0.39 to 0.51	(67 to 87lb)

The water/cement ratio was varied when necessary in order to main-

tain a minimum slump of 3in., and at the same time to attain a strength bordering 5000psi.. A workable mix was required due to congestion of reinforcement between and underneath each web opening.

Concrete cylinder strengths varying from 3700 to 5580 psi were obtained; these figures were based on the average of two cylinder strength per batch. A third cylinder per batch gave the splitting tensile strength which varied from 292 to 513 psi.. Table A.3 presents the age at test, compressive strength and the tensile concrete strength.

d) Prestressing strand

The prestressing strand used in the test beams was 250K grade, 7-wire stand, with a 3/8 in. nominal diameter and complied with ASTM-A-416 Specifications. The stress-strain diagram as obtained from tensile tests is shown in Fig.A.1.

e) Shear reinforcement

The stirrups were made from #3 deformed bars, bent to a specified shape by the supplier. The stress-strain curves for this reinforcement is shown in Fig. A.2. These curves were obtained from tensile tests conducted on a Baldwin Testing Machine. The strains were obtained from electrical resistance strain gages mounted on the bars. As shown in Fig. A.2, the curves are of use up to the yield point of the steel but due to malfunction of the gages, are of no use above the yield point.

f) Longitudinal reinforcement

The longitudinal reinforcement consisted of #3 or #4 deformed bars. The stress-strain diagram of these is shown in Fig. A.2.

A.2 Fabrication

a) Formwork

The forms used in this test progam were fabricated in three sections, each 8 ft. long. They were slip type forms of 3/4 in. and 1/2 in. plywood with timber battens to maintain the correct beam shape under the pressure of fresh concrete. Figure A,3 shows a X-section of the forms at an opening. As previously stated,the forms were of the slip type; the exterior portion served to form the flanges and to confine the interior 1/2 in. plywood web forms, while the latter served to keep the styrofoam shaping blocks in position and to provide a good web finish. The styrofoam blocks were used to shape the openings. There was no mechanical connection between the exterior and interior forms. The exterior forms were however bolted to a steel channel base form. Figure A.4 shows the forms with one side removed.

After the concrete had set for 24 hours, the exterior forms were unbolted and slipped off wkile the interior forms were peeled off the hardened concrete. The styrofoam blocks were then punched through. This method worked extremely well and is recommended for future casting operations.

b) Prestressing operation

The prefabricated shear reinforcement cages were laid

on the base of the formwork with one side removed for accessibility. High strength prestressing cables were then threaded through the prestressing abutments, form end-plates and through the shear cages. At one end of the beam, prefabricated steel dynamometers were slipped on the strands and CCL anchoring devices were then used to grip the cables. Allsteel abutments were designed as special equipment and resisted the tensioning forces by friction and by bolt shear, as shown in Fig. A.4.

Each strand was tensioned individually using a Simplex centre hole hydraulic jack operated by an electrically driven Blackhawk hydraulic pump; prestressing equipment is illustrated in Fig. A.5. The correct tensioning loads were obtained by reading the strains of the dynamometers. Strains were measured by electrical resistance strain gages mounted on the dynamometers. When the correct prestress was achieved, a CCL grip, located between the jack and the abutments, was pushed snug to the abutment back-plate; the jack was then released and removed. After all the strands were tensioned, the form side was erected, adjusted for alignment and then five steel wood clamps were placed along the top of the forms to maintain a correct flange width.

c) Casting and curing

The concrete was mixed in a nine cubic foot vertical drum mixer. Each beam required three batches except for beam #1 which required four. The concrete was compacted using an electric immersion type vibrator. Compaction control was carefully supervised so that no air pockets formed beneath the styrofoam blocks.

For each batch of concrete, three 6in. x 12in. control cylinders were cast; two were tested in compression and one in splitting.

Immediately after casting and finishing, each beam was covered by a damp-proof sheet, in an attempt to minimize early shrinkage cracking and to promote curing. The side forms were removed after 24 hours, the beams were then enclosed in a saturated burlap and then recovered by the damp-proof sheet. A humid atmosphere was maintained for five to six days prior to the release of the strands. The control cylinders were always subjected to the same curing conditions as the beams. After release of the strands, the beams were stored in laboratory atmosphere for periods of 17 to 28 days before testing.

d) Release of prestress

The prestress was released in beams #1 to 6 inclusive after six days of moist curing and in beams #7 to 9 after five days. The first stage in the release was to cut the strands at one end of the beam. This was accomplished by gently applying heat from an oxy-acetylene flame, over a length of about four feet between the stopend and the abutment, until the individual strands broke. Fracture was always gentle, indicating a uniform transfer of prestress had resulted.

A.3 Prestress losses

A complete set of Demec strain gage readings was taken immediately before and after release and these were used, together

with the initial readings taken at the beginning of each test to calculate losses of strain in the beam and strands.

Losses which occurred during the prestressing operation, due to anchorage slip and to release of the jack, all took place before the initial prestress was calculated. Therefore,the losses which are presented under the "Total loss" heading in Table A.4 refer to elastic, creep, and shrinkage losses in the concrete and to relaxation loss in the steel.

The initial prestress forces in the strands were calculated from the dynamometer readings taken immediately before release.

A.4 Loading apparatus

Two concentrated loads were applied by means of two 220k Amsler hydraulic jacks. The loading frame and jack arrangements are shown in Fig. A.6. Equal loading at two points was assured by coupling the hydraulic jacks in parallel. During each increment, the load was maintained manually.

The loads were applied through 6 in. diameter steel bearing plates which were seated to the correct level on plaster of Paris. Both supports were hinged so as to permit rotation; one of the supports was longitudinally fixed and the other was mounted on rollers to permit simple beam action.

TABLE A.1

SIEVE ANALYSIS OF SAND

SIEVE SIZE	WEIGHT RETAINED (gms)	% RETAINED	CUMULATIVE % RETAINED	A.S.T.M. STANDARD
#4	17.5	3.0	3.0	0 - 5
#8	85.2	14.7	17.7	-
#16	54.6	9.5	27.2	20 - 55
#30	60.0	10.3	37.5	-
#50	208.4	35.8	73.3	70 - 90
#100	122.9	21.1	94.4	90 - 98
Pan	17.8	3.1	-	-
Silt	14.4	2.5	-	-
Total	580.8	100.0		
Fineness Modulus		2.53		

TABLE A.2

SIEVE ANALYSIS OF COARSE AGGREGATE

SIEVE SIZE	% RETAINED	CUMULATIVE % RETAINED
3/4	0	0
3/8	5.9	5.9
#4	87.1	93.0
Pan	7.0	100.0
Total	100.0	

TABLE A.3

SUMMARY OF CONCRETE STRENGTHS

BEAM No.	MIX No.	AGE AT TEST (DAYS)	AVGE. CYLINDER STRENGTH (PSI)	AVGE. SPLITTING STRENGTH (PSI)
1	I	21	4710	420
	II		4655	429
	III		4725	389
	IV		4655	443
2	I	21	4850	415
	II		4990	292
	III		4900	415
3	I	28	5200	486
	II		4475	415
	III		4715	425
4	I	21	3700	380
	II		4775	425
	III		4400	478
5	I	21	4875	460
	II		4760	415
	III		5130	354
6	I	17	5020	425
	II		4510	460
	III		5020	513
7	I	16	4725	424
	II		5080	477
	III		4805	397
8	I	18	4895	486
	II		4595	381
	III		4980	389
9	I	18	5480	416
	II		5090	412
	III		4885	403

TABLE A.4

SUMMARY OF PRESTRESS LOSSES

BEAM #	INITIAL PRESTRESS (ksi)	ELASTIC LOSS (ksi)	TIME LOSS* (ksi)	TOTAL LOSS (ksi)	Pe** (ksi)
1	169.2	14.05	32.15	46.2	123.0
2	171.0	19.4	32.4	51.8	119.2
3	174.8	21.1	43.9	65.0	109.8
4	178.5	22.2	48.4	70.6	107.9
5	179.0	18.8	42.2	61.0	118.0
6	178.5	27.8	41.5	69.3	109.2
7	177.5	21.9	43.1	65.0	112.5
8	177.5	19.4	42.6	62.0	115.5
9	176.0	18.0	40.0	58.0	118.0

* measured from time of transfer to time of testing
** effective prestress

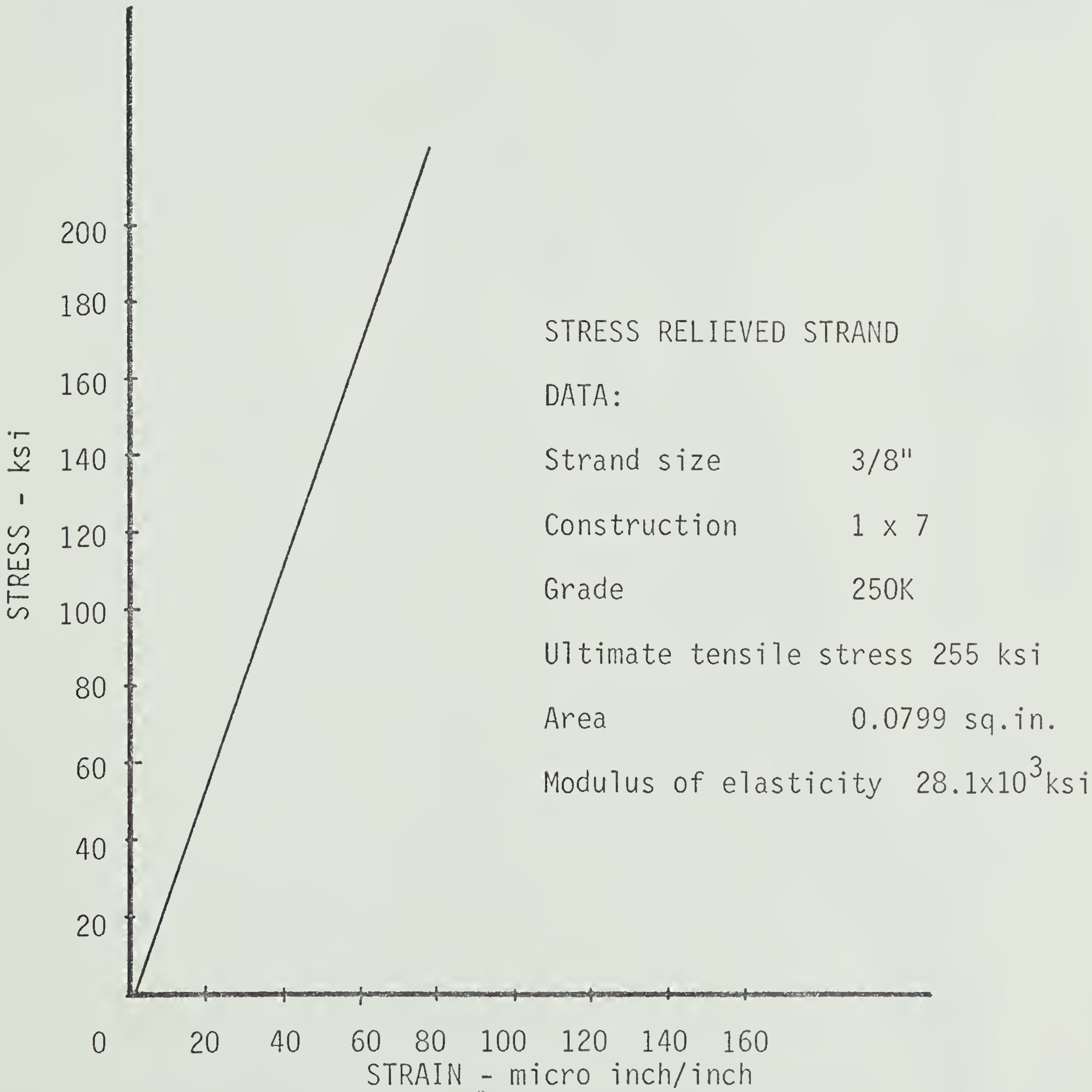


FIGURE A.1 STRESS - STRAIN RELATIONSHIP OF PRESTRESSING STRAND

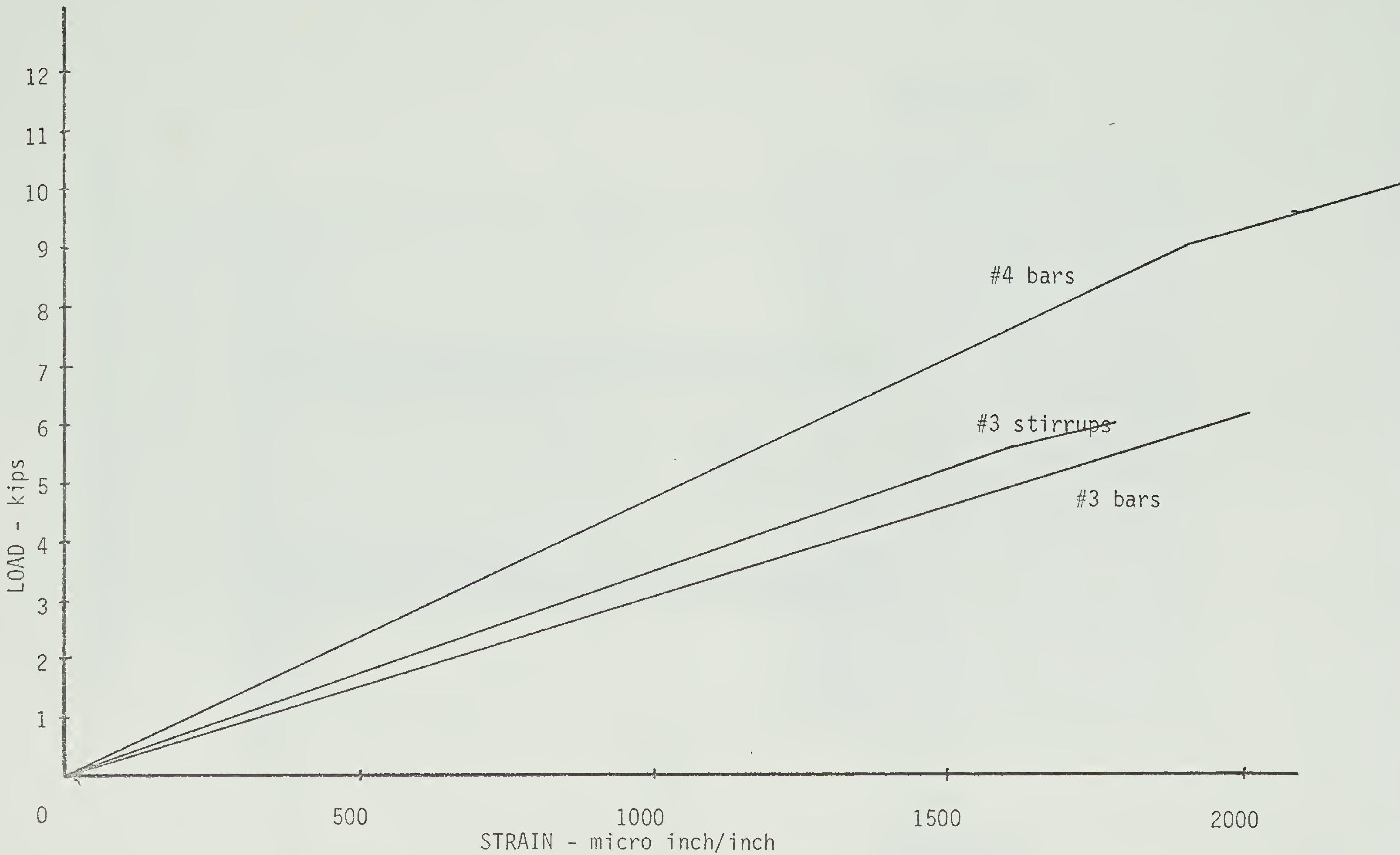


FIGURE A.2 LOAD - STRAIN RELATIONSHIPS FOR VERTICAL AND LONGITUDINAL REINFORCEMENT

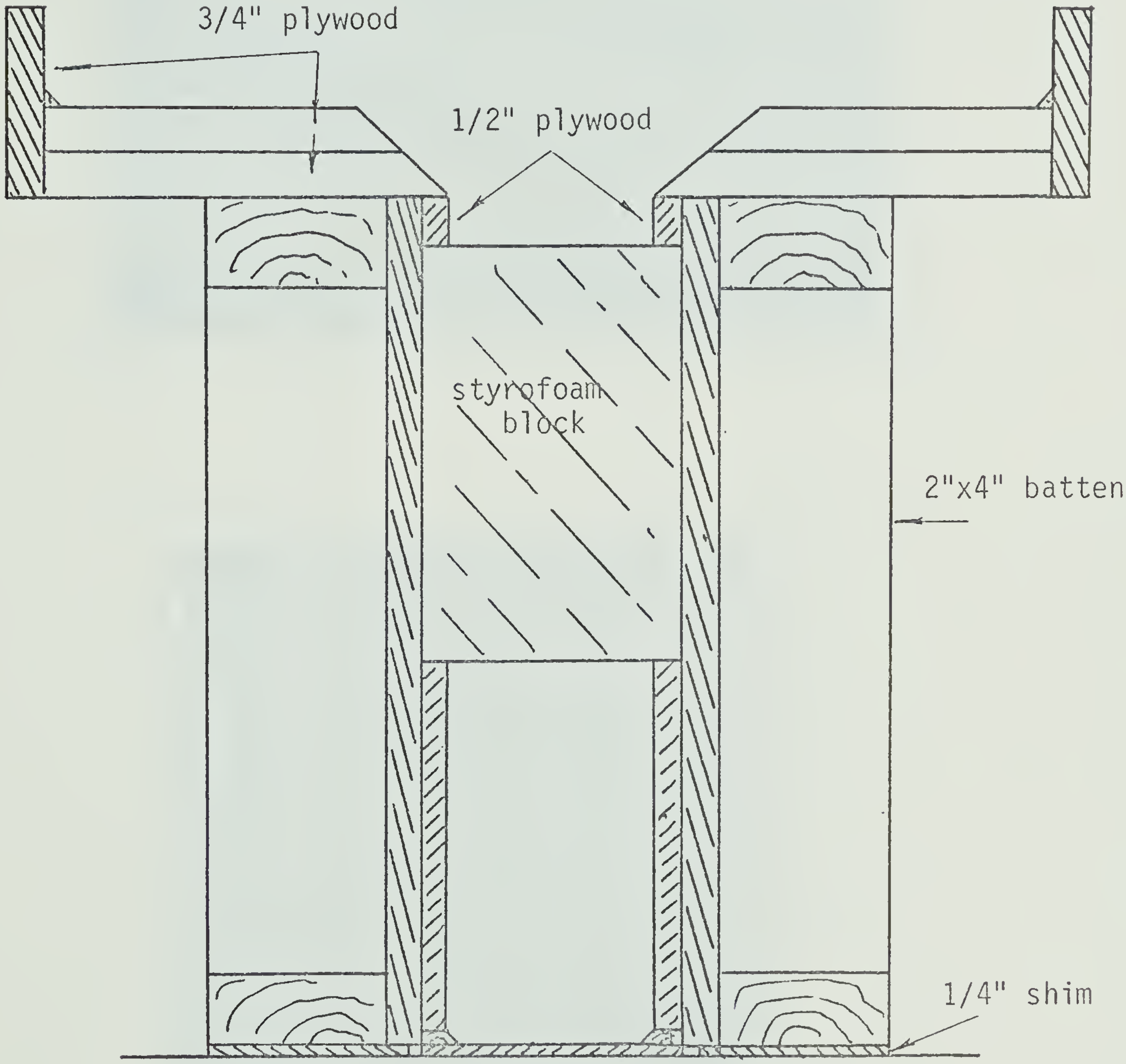
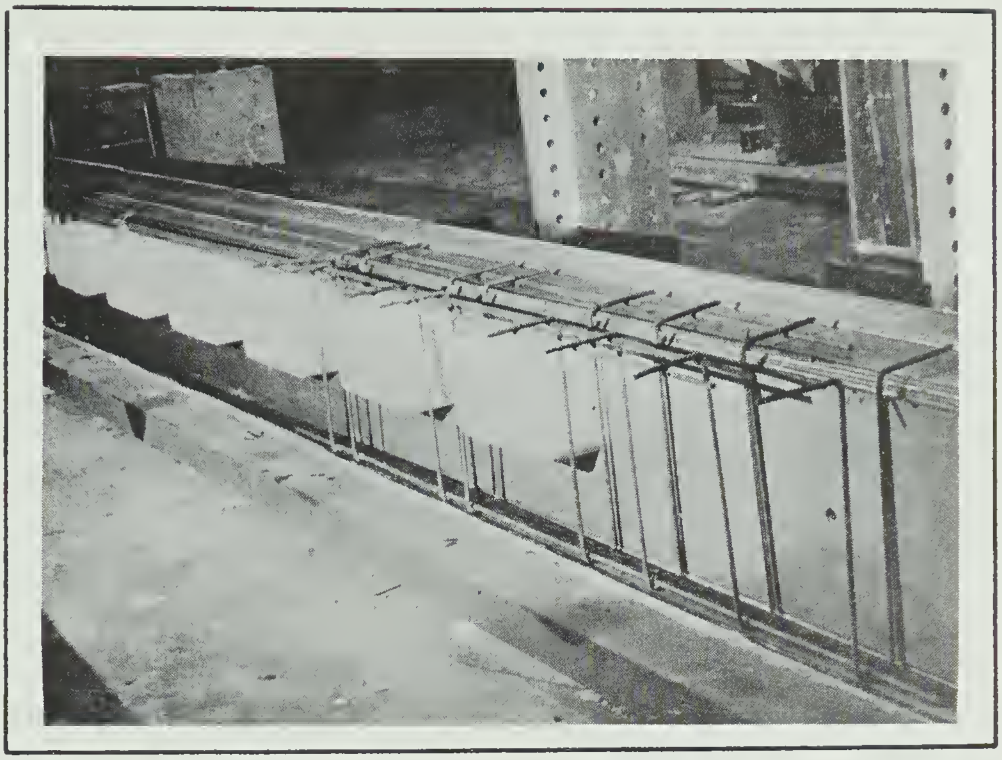


FIGURE A.3 TYPICAL X-SECTION THROUGH FORMWORK



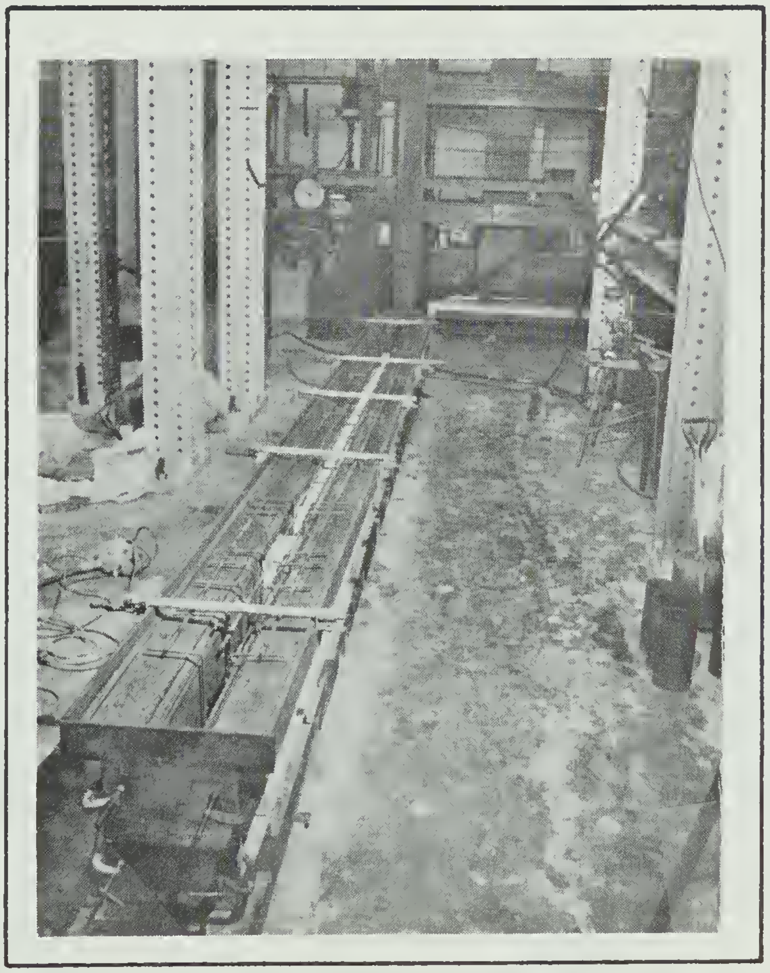


FIGURE A.4 FORMWORK

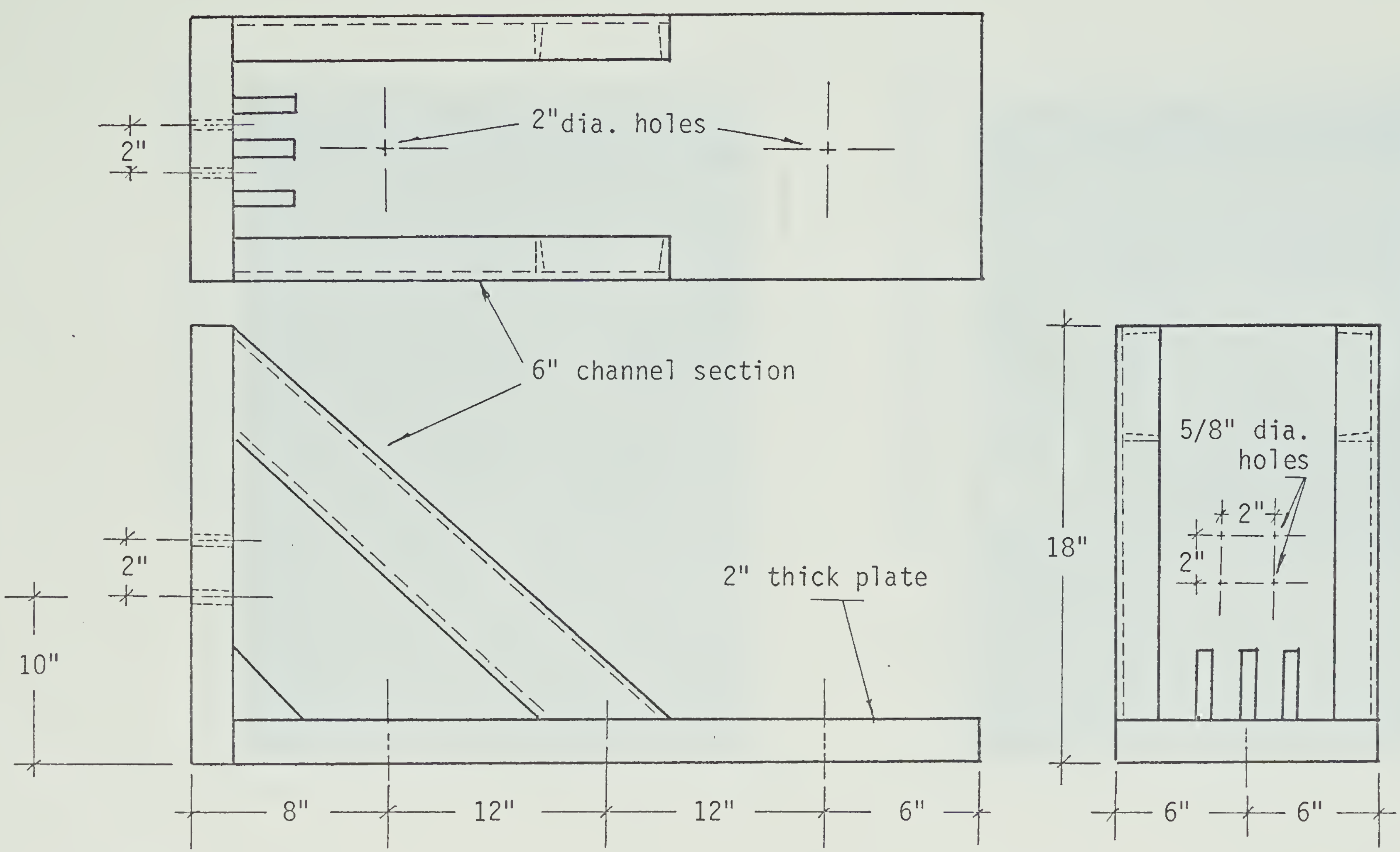


FIGURE A.5 ALL STEEL PRESTRESSING ABUTMENTS

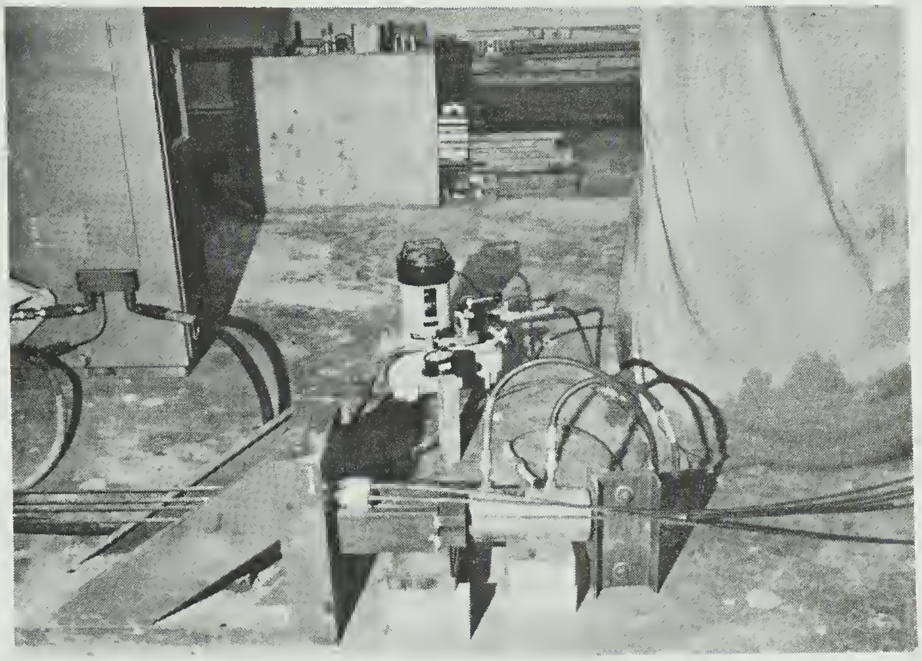


FIGURE A.6 PRESTRESSING EQUIPMENT

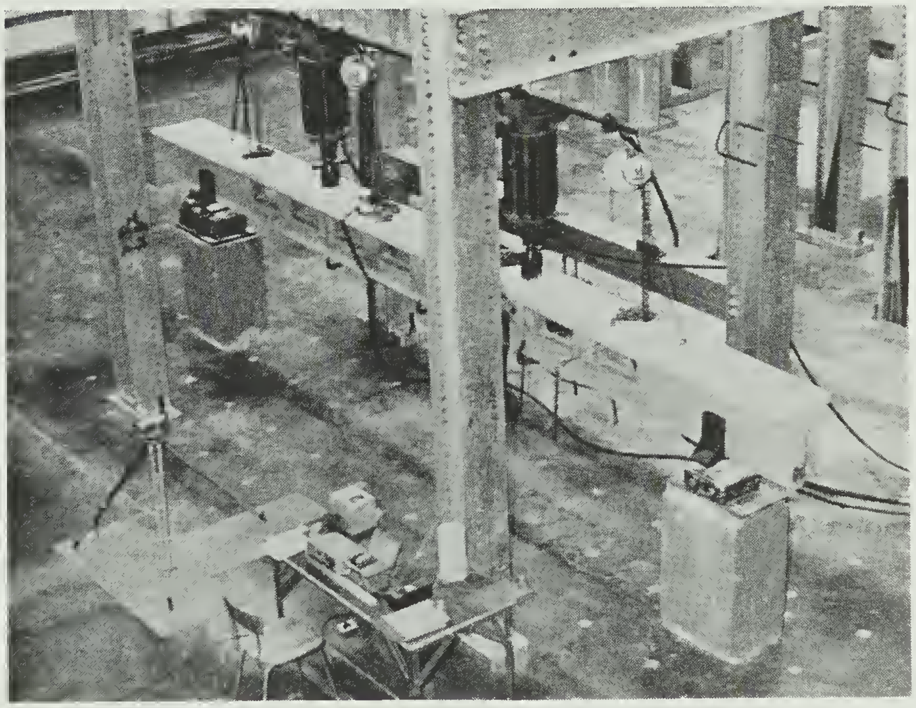
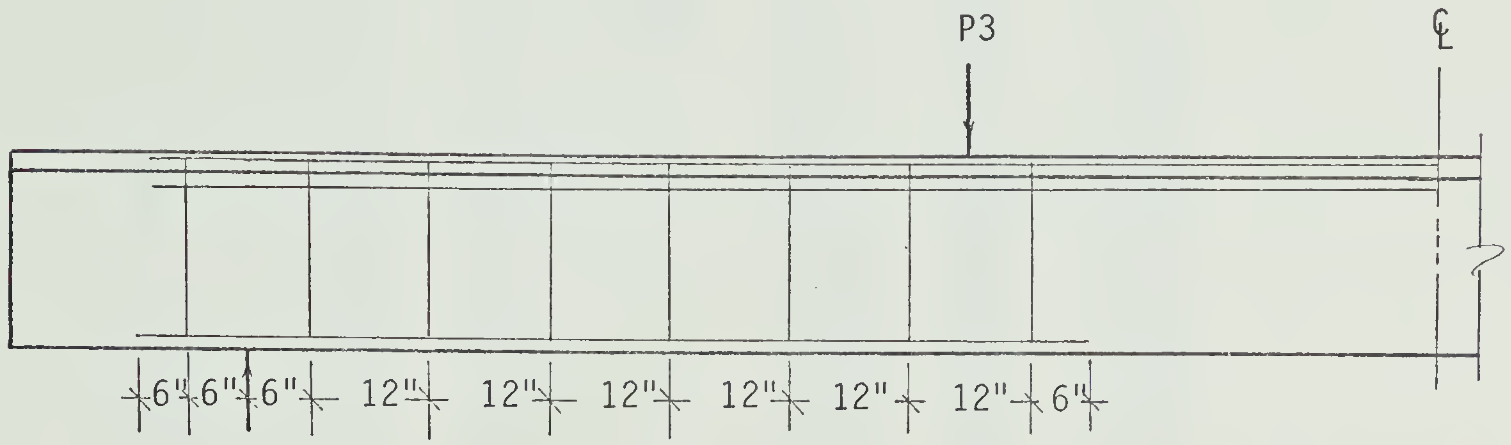


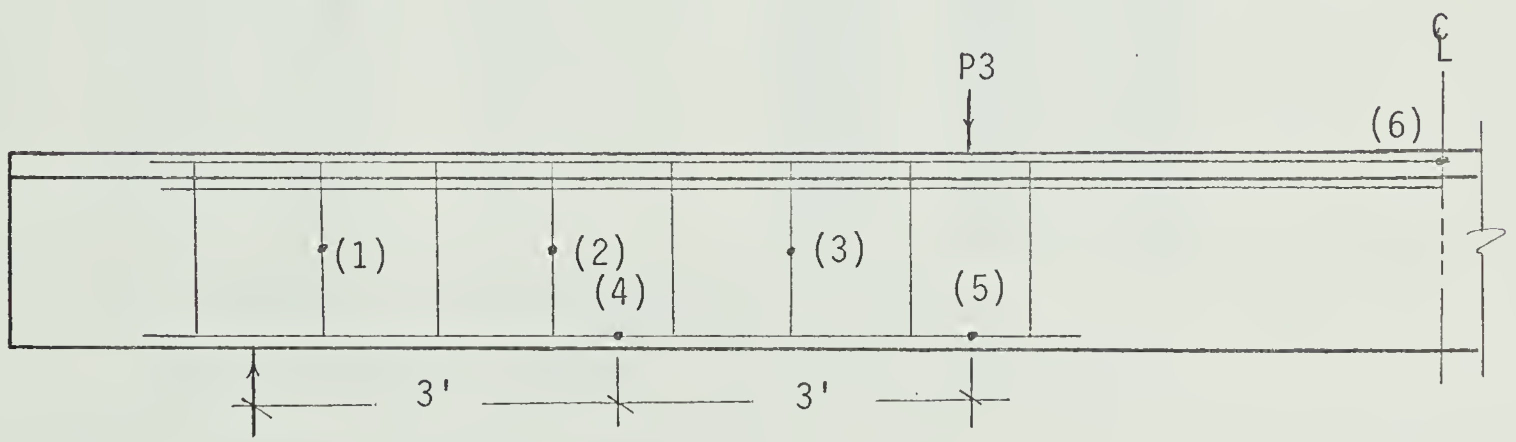
FIGURE A.7 TYPICAL TEST SET-UP

APPENDIX B

DATA



BEAM #1 REINFORCEMENT DETAIL



BEAM #1 STRAIN GAGE LOCATIONS

FIGURE B.1.1 BEAM #1

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **
0	0	0	0	0	0	0
1	-4*	-3	N.O.	+8	+14	-16
2	-4	-8		+6	+25	-30
3	-6	-2		+42	+96	-100
4	-6	-2		+55	+135	-148
5	-10	-6		+66	+166	-188
6	-10	-6		+85	+207	-228
7	-10	-2		+100	+250	-266
8	-15	-5		+110	+278	-308
9	-23	-14		+126	+325	-346
9.5	-24	-12		+125	+320	-384
10.0	-22	-14		+133	+354	-403
10.5	-22	-12		+142	+374	-433
11.0	-24	-10		+148	+390	-452
11.5	-26	-10		+160	+402	-495
12.0	-23	-10		+166	+447	-520
12.6	-26	-10		+181	+500	-558
13.1	-29	-18		+183	+560	-626
13.6	-28	-18		+190	+890	-644
14.2	-40	-24		+196	+1376	-683
14.5	-34	-22		+204	+1482	-690
15.1	-32	-18		+214	+1856	-708
15.7	-32	-18		+226	+2054	-732
16.3	-34	-16		+235	+2200	-760
16.9	-30	-18		+245	+2400	-784
17.5	-40	-10		+264	+2110	-794
18.1	-38	-6		+280	+2138	-780
18.7	-34	-4		+288	+2208	-782

20.3 FAILURE - Visible rupture of one strand.

* NOTE: + indicates tension, - indicates compression

** Measurements in micro inches/inch

TABLE B.1.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(16)	(22)	(26)	(29)
i	0	0	0	0
ii	-5 *	+70	+29	+47
1	+10	+41	+69	+94
2	16	44	67	89
3	18	43	63	85
4	20	113	60	79
5	25	44	58	74
6	28	45	57	69
7	32	46	52	64
8	35	46	49	56
9	39	47	46	46
9.5	42	47	43	34
10.0	45	49	39	27
10.5	47	49	36	21
11.0	50	49	33	9
11.5	57	51	-2	-47
12.0	62	43	-24	-76
12.6	71	38	-34	-126
13.1	84	25	-65	-182
13.6	89	13	-87	-235
14.2	98	-3	-122	-260
14.5	102	-9	-136	-318
15.1	109	-29	-172	-390
15.7	118	-50	-218	-480
16.3	129	-77	-273	-606
16.9	143	-116	-352	----

20.3 FLEXURAL FAILURE

* NOTE: x 10^{-4} inches

TABLE B.1.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.02	.02
2	.04	.05	.04
3	.07	.08	.06
4	.09	.11	.08
5	.11	.14	.11
6	.14	.17	.13
7	.16	.20	.15
8	.19	.23	.18
9	.22	.26	.19
9.5	.23	.28	.21
10.0	.25	.30	.23
10.5	.28	.34	.29
11.0	.30	.37	.31
11.5	.35	.43	.40
12.0	.41	.51	.50
12.6	.49	.61	.61
13.1	.62	.78	.69
13.6	.70	.88	.84
14.2	.84	1.07	.99
14.8	1.00	1.26	1.05
15.1	1.07	1.35	1.25
15.7	1.27	1.59	1.35
16.0	1.38	1.74	1.61
16.6	1.65	2.06	2.03
17.2	2.07	2.58	2.30
17.5	2.35	2.92	3.39
18.1	3.42	4.08	-----
18.7	3.94	4.37	
19.0	4.29	4.52	

20.3 FAILURE

* indicates 1/3 point

TABLE B.1.3 DEFLECTIONS

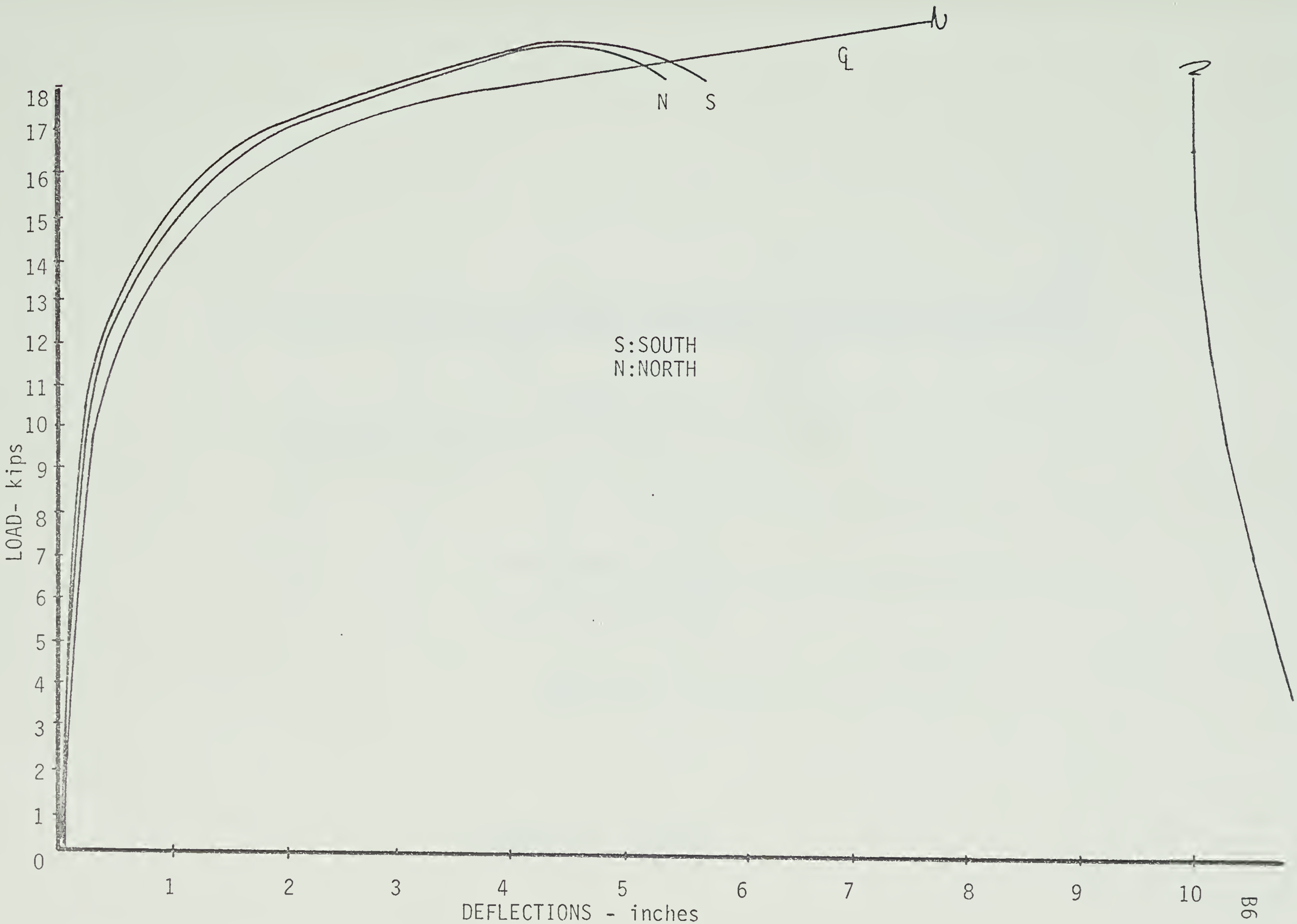
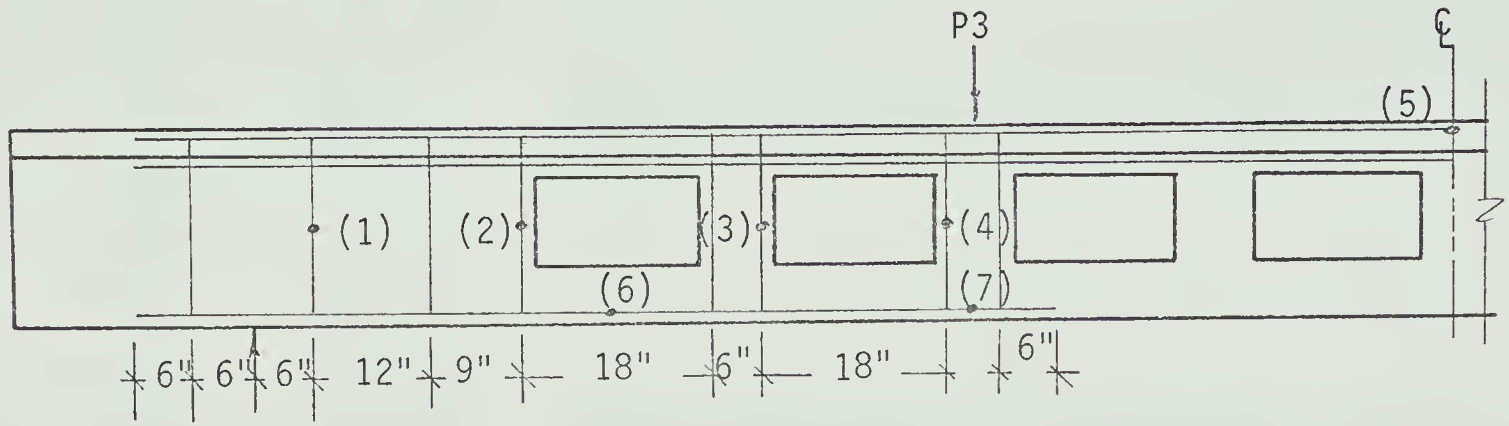


FIGURE B.1.2 LOAD-DEFLECTION DIAGRAM,BEAM #1



REINFORCEMENT DETAIL

STAIN GAGE LOCATIONS

BEAM #2

FIGURE B.2.1 BEAM #2

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **
0	0		0	0	0
1	0		+4	-23	-24
2	-4 *		30	- 30	- 43
3	- 5		60	- 44	- 66
4	- 7		130	- 60	- 90
5	- 8		306	- 77	- 116
6	- 12		540	- 97	- 140
7	- 15		736	- 120	- 166
8	- 18		937	- 134	- 196
9	- 18		1153	- 142	- 222
9.5	- 18		1232	- 146	- 240
10.0	- 17		1340	- 166	- 254
10.5	- 19		1426	- 148	- 284
11.0	- 20		1462	- 130	- 302
11.5	- 22		1480	- 114	- 328
12.0	- 16		1492	- 95	- 356
12.5	- 16		1500	-40	- 386
13.0	- 20		1512	+70	- 410
13.5	- 16		160	+702	- 223

* NOTE: + indicates tension,- indicates compression

** Measurements in micro inch/inch

TABLE B.2.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(16)	(22)	(26)	(27)	(28)
i	0	0	0	0	0
ii	-2*	+70	+19	+40	+58
0	+23	+27	+57	+92	+120
1	+26	+41	+55	+89	+116
2	28	44	55	87	114
3	31	43	53	85	109
4	33	43	51	81	105
5	37	44	48	80	103
6	39	45	47	77	98
7	43	46	44	73	93
8	47	46	43	68	88
9	48	47	42	67	84
9.5	50	47	41	65	80
10.0	53	49	39	62	75
10.5	58	49	39	59	74
11.0	60	49	36	55	70
11.5	65	51	38	54	65
12.0	70	43	36	49	59
12.5	76	38	26	31	39
13.0	83	25	22	20	23
13.5 FAILURE				65	83

* NOTE: x 10^{-4} inches

i indicates before release of strands
ii indicates after release of strands

TABLE B.2.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.03	.02
2	.04	.05	.04
3	.07	.08	.06
4	.09	.11	.09
5	.12	,14	.11
6	.15	.18	.14
7	.18	.21	.18
8	.21	.25	.21
9	.25	.30	.25
9.5	.28	.33	.29
10.0	.33	.40	.35
10.5	.37	.43	.37
11.0	.42	.51	.43
11.5	.50	.61	.51
12.0	.62	.75	.63
12.5	.74	.92	.75
13.0	.85	1.07	.97
13.5	1.96	1.57	1.04

* indicates 1/3 point

TABLE B.2.3 DEFLECTIONS

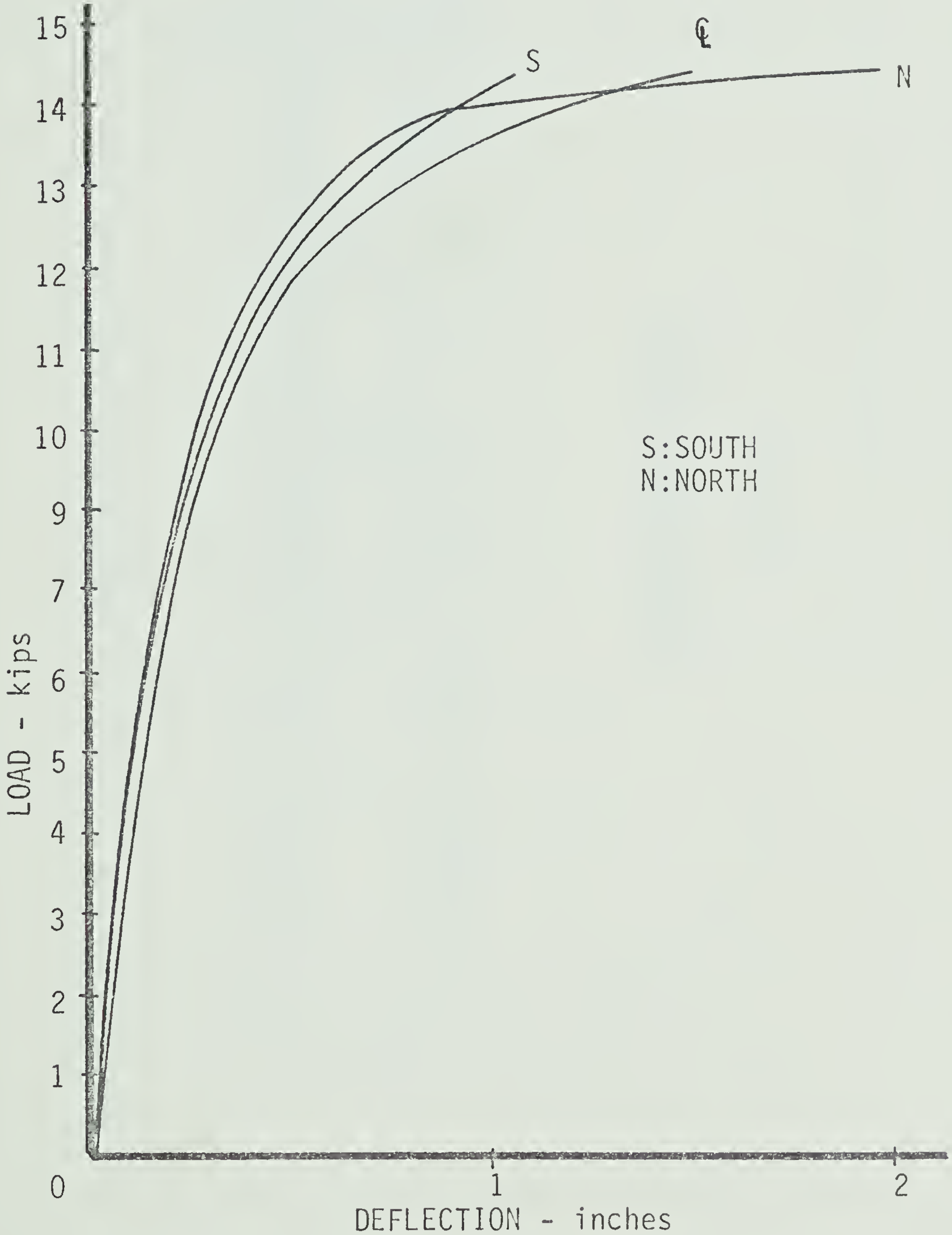
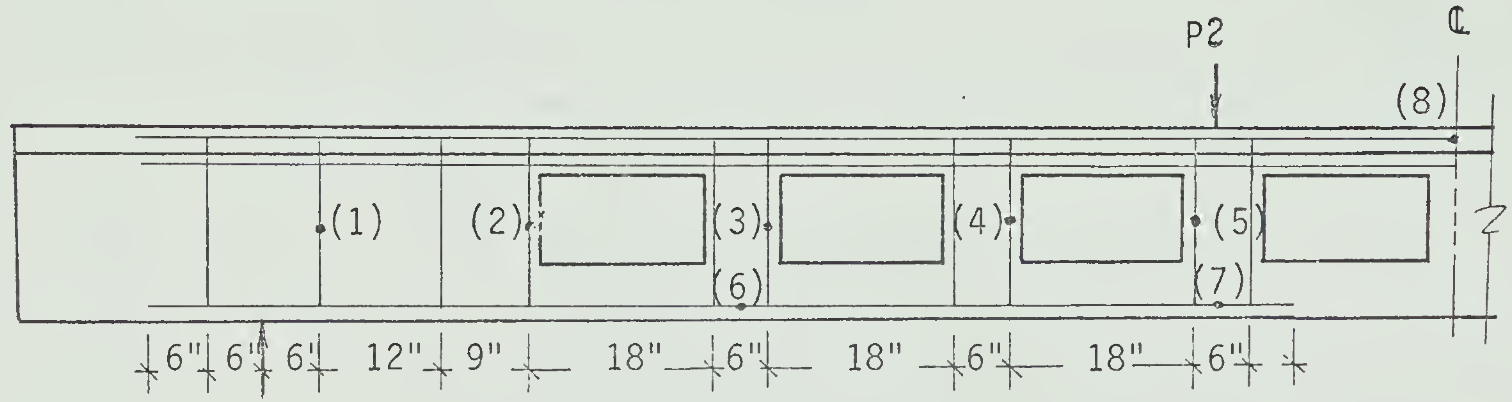


FIGURE B.2.2 LOAD-DEFLECTION DIAGRAM,BEAM #2



REINFORCEMENT DETAIL
STRAIN GAGE LOCATIONS

BEAM #3

FIGURE B.3.1

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **	(7) **	(8) **
0	0	0	0	0	0	0	0	0
1	+6 *	+7	+8	N.O.	N.O.	+20	+60	-26
2	0	20	16			39	123	-60
3	-2	52	75			51	192	-92
4	-2	106	342			70	258	-122
5	-5	330	648			88	355	-152
6	-9	515	868			103	518	-190
7	-10	670	1062			119	790	-214
8	-15	850	1290			121	1156	-260
9	-12	1030	1595			155	1690	-298
9.5	-14	1084	1670			173	1954	-314
10.0	-10	1149	1760			200	2230	-326
10.5	-10	1206	1850			233	2464	-342
11.0	-16	1270	1960			308	2700	-347
11.5	-12	1322	2070			364	2920	-355
12.0	-20	1392	2192			445	2972	-356
12.5	-20	1445	2296			526	3020	-356
13.0	-17	1530	2382			702	2980	-335
13.5	-16	1648	2537			934	3008	-278

* NOTE: + indicates tension, - indicates compression

** Measurements in micro inch/inch

TABLE B.3.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(3)	(2)
i	0	0	0	0	0
ii	-5*	+15	+19	+39	+54
0	+34	+45	+67	+102	
1	+39	47	+66	98	+122
2	42	47	60	92	115
3	48	48	60	90	111
4	52	49	58	87	105
5	55	52	55	83	99
6	60	53	54	80	94
7	63	56	51	76	86
8	70	52	46	56	46
9	88	31	21	29	-24
9.5	99	19	-4	19	-41
10.0	107	8	-22	7	-64
10.5	114	2	-59	-10	-100
11.0	125	-16	-96	-29	-137
11.5	134	-26	-127	-47	-185
12.0	144	-42	-184	-76	-260
12.5	153	+44	-233	-108	-340
13.0	169	+22	-280		-459
13.5					
13.95 FAILURE					

* NOTE: x 10^{-4} inches, (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.3.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.03	.03
2	.05	.06	.06
3	.08	.10	.08
4	.11	.14	.12
5	.14	.19	.15
6	.18	.23	.19
7	.22	.29	.24
8	.31	.37	.32
9	.48	.66	.51
9.5	.60	.80	.60
10.0	.70	.93	.70
10.5	.82	1.09	.80
11.0	.95	1.26	.94
11.5	1.07	1.44	1.06
12.0	1.26	1.71	1.24
12.5	1.44	1.95	1.42
13.0	1.71	2.33	1.71
13.5	2.21	3.06	2.19
13.95	----	4.01	----

* indicates 1/3 point

TABLE B.3.3 DEFLECTIONS

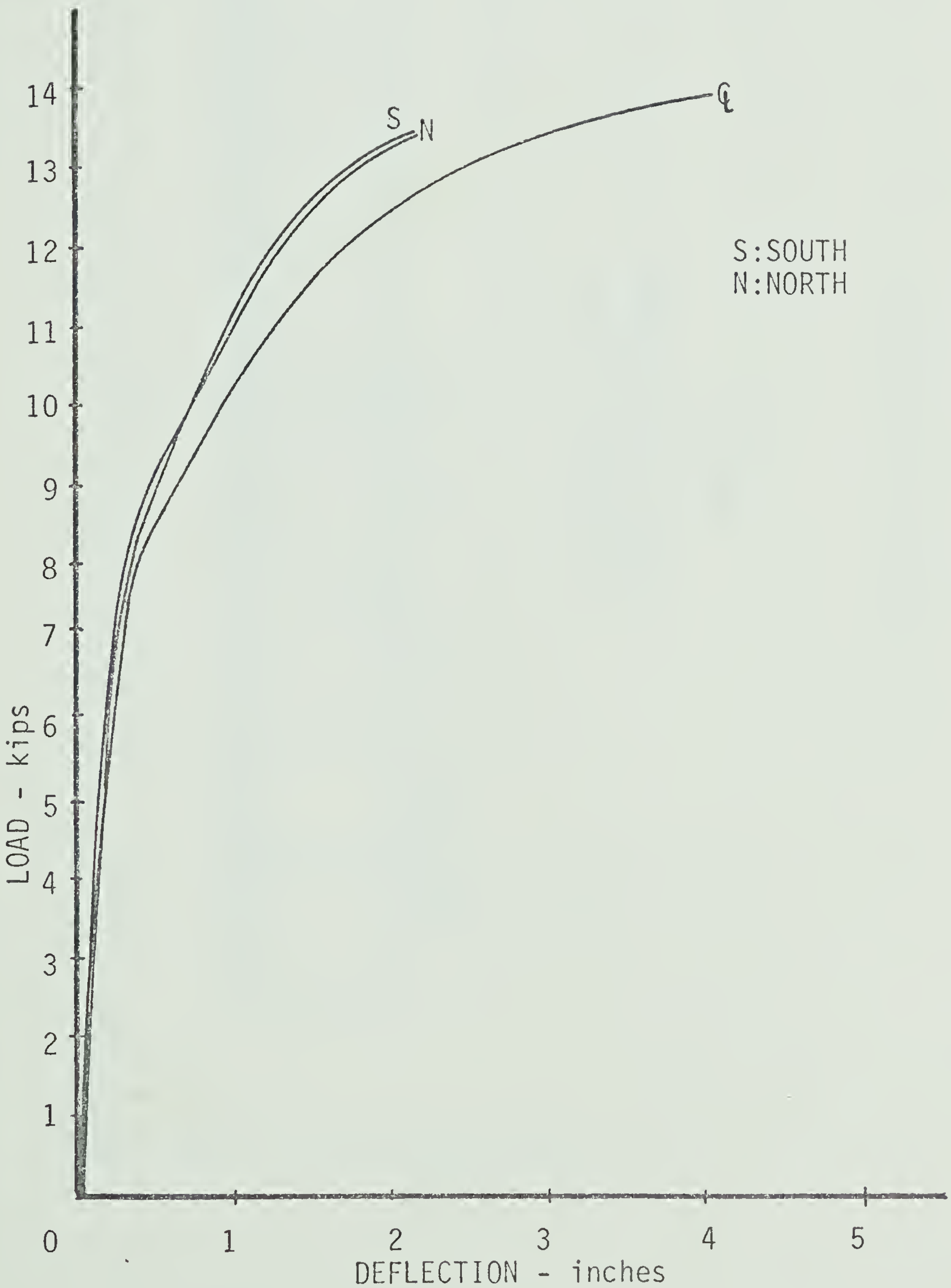
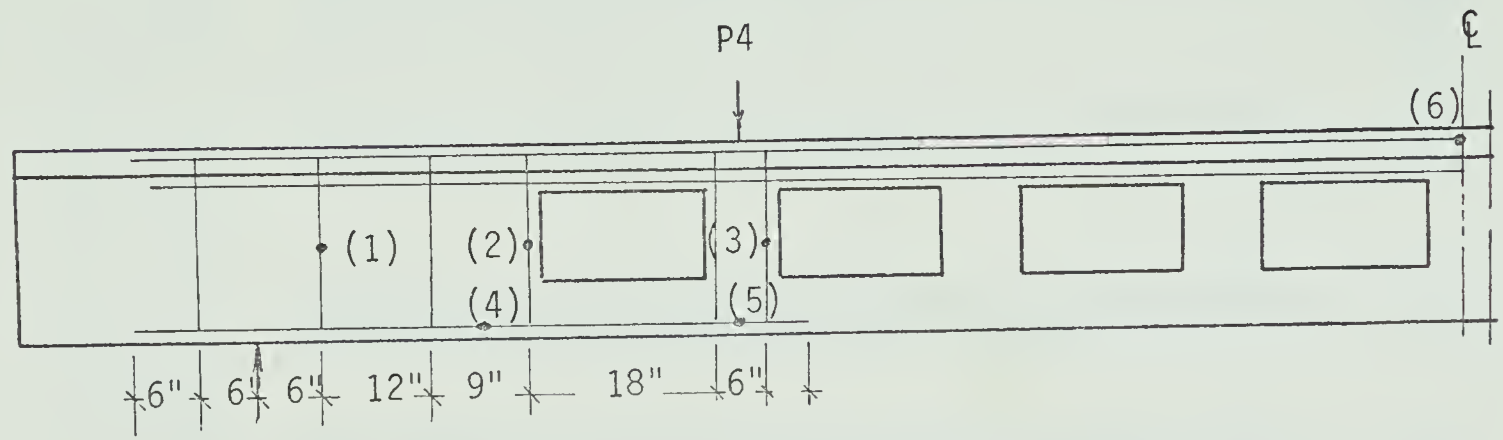


FIGURE B.3.2 LOAD-DEFLECTION DIAGRAM, BEAM #3



REINFORCEMENT DETAIL

STAIN GAGE LOCATIONS

BEAM #4

FIGURE B.4.1 BEAM #4

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **
0	0	0	0	0	0	0
1	-2 *	+8	-6	+8	+30	-16
2	- 4	14	- 22	16	64	- 32
3	- 4	22	- 36	14	92	- 54
4	- 4	46	- 48	22	136	- 74
5	- 6	74	- 68	26	156	- 98
6	- 8	120	- 82	26	202	- 120
7	- 11	157	- 100	30	250	- 142
8	- 11	216	- 115	28	288	- 162
9	- 13	379	- 135	32	322	- 187
9.5	- 14	412	- 140	23	358	- 197
10.0	- 15	455	- 144	23	389	- 211
10.5	- 15	498	- 148	23	405	- 221
11.0	- 16	537	- 157	28	449	- 233
11.5	- 16	596	- 158	28	498	- 245
12.0	- 17	630	- 167	33	546	- 257
12.5	- 15	692	- 169	33	585	- 266
13.0	- 15	730	- 170	37	648	- 279
13.5	- 14	795	- 170	30	710	- 290
14.0	- 15	843	- 160	32	788	- 301
14.5	- 15	882	- 154	40	857	- 312
15.0	- 15	925	-154	46	960	- 327
15.5	- 14	1018	+655	41	1026	- 340
16.0	- 16	1075	776	48	1146	- 364
16.5	- 12	1134	851	46	1236	- 382
17.0	- 13	1173	906	71	1361	- 398

*NOTE: + indicates tension, - indicates compression

** Measurements in micro inch/inch

TABLE B.4.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(2)	(1)
i	0	0	0	0	0
ii	-2*	+15	+210	+54	+73
0	+33	+36	+68	+140	+158
1	+33	37	+67	137	+154
2	36	37	67	135	151
3	39	37	64	132	147
4	40	38	64	130	145
5	43	40	64	127	141
6	45	41	63	124	138
7	49	42	63	121	135
8	50	43	61	118	130
9	52	44	60	115	127
9.5	53	45	61	115	126
10.0	55	45	60	113	123
10.5	57	45	59	110	120
11.0	58	46	59	110	119
11.5	59	47	57	108	117
12.0	60	47	57	107	115
12.5	60	47	57	104	113
13.0	63	47	55	103	111
13.5	66	48	55	101	108
14.0	68	48	54	98	105
14.5	66	48	55	97	102
15.0	69	49	53	95	101
15.5	69	49	51	92	98
16.0	72	46	52	90	98
16.5	73	45	50	90	98
17.0	--	44	50	89	98

* NOTE: x 10^{-4} inches (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.4.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.02	.02
2	.04	.05	.03
3	.06	.07	.05
4	.08	.09	.07
5	.10	.11	.09
6	.12	.14	.11
7	.14	.16	.13
8	.16	.19	.16
9	.19	.21	.18
9.5	.20	.23	.19
10.0	.22	.25	.21
10.5	.23	.26	.22
11.0	.24	.28	.23
11.5	.26	.29	.25
12.0	.27	.31	.26
12.5	.29	.33	.28
13.0	.31	.35	.29
13.5	.33	.37	.31
14.0	.35	.39	.33
14.5	.37	.41	.37
15.0	.40	.44	.40
15.5	.44	.49	.44
16.0	.51	.58	.52
16.5	.57	.65	.58
17.0	.68	.78	.67
17.5	----	.80	----

* indicates 1/3 point

TABLE B.4.3 DEFLECTIONS

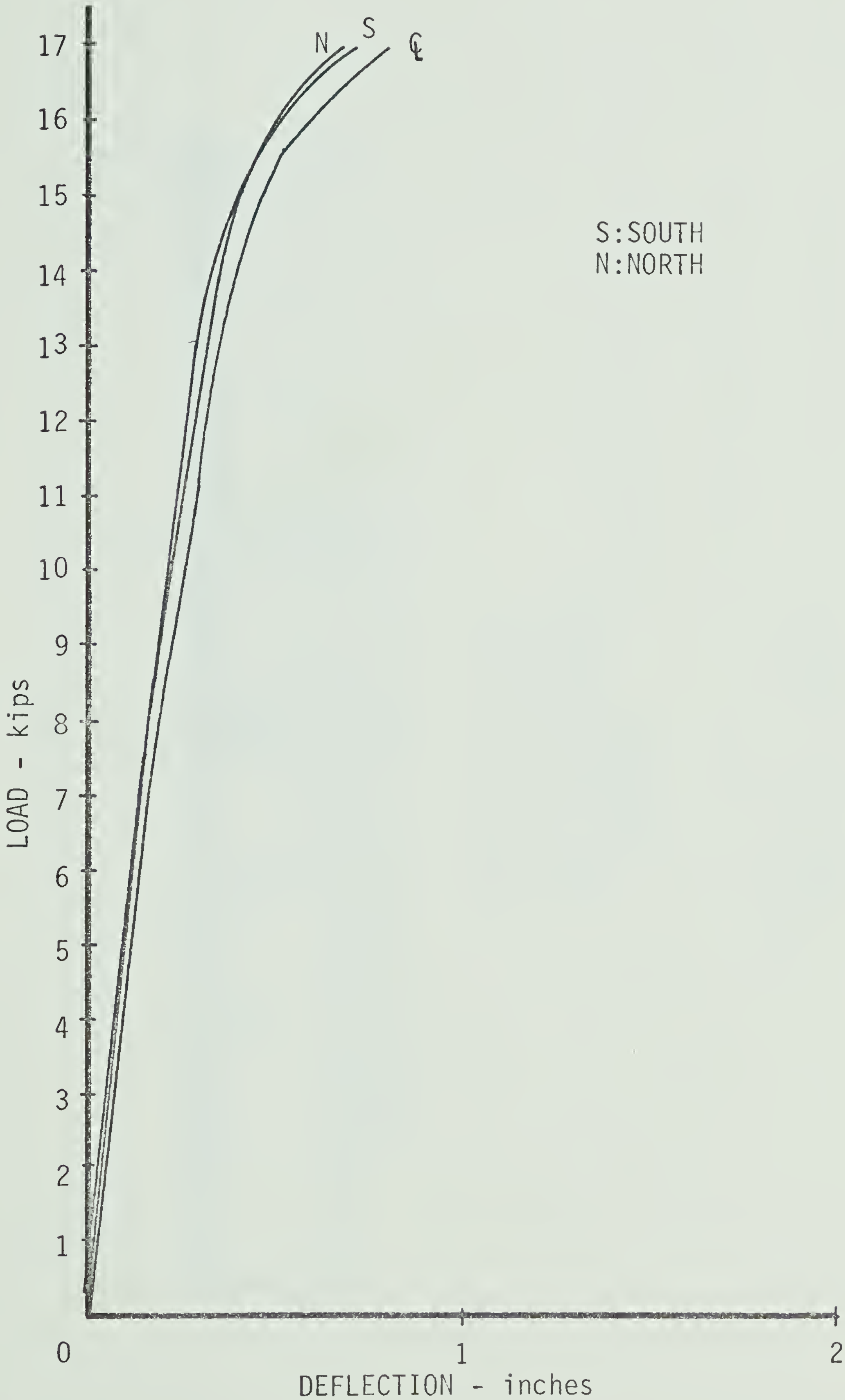
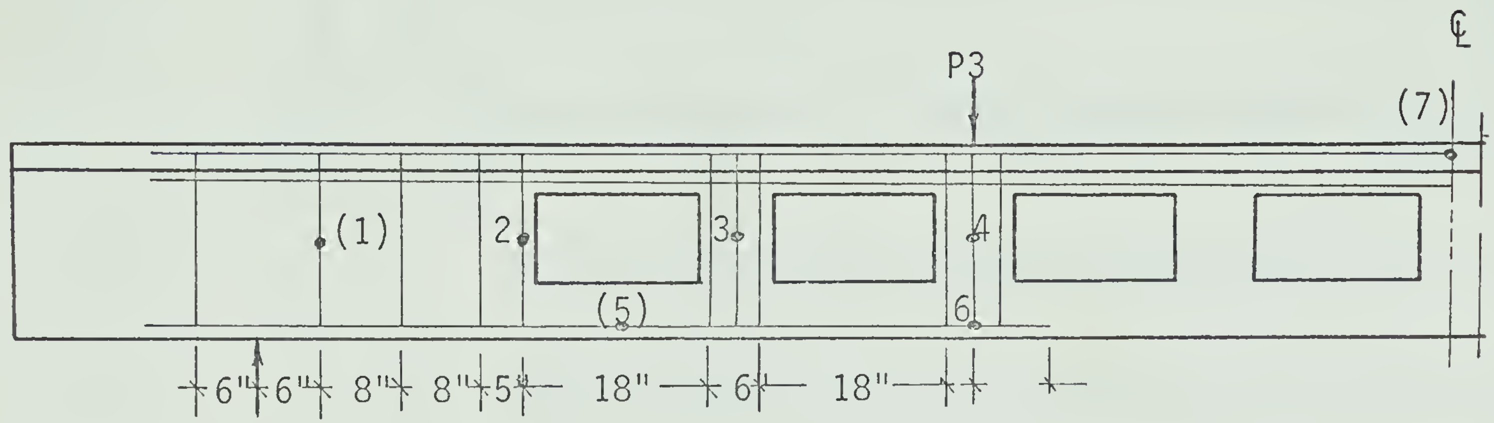


FIGURE B.4.2 LOAD-DEFLECTION DIAGRAM, BEAM #4



REINFORCEMENT DETAIL

STAIN GAGE LOCATIONS

BEAM #5

FIGURE B.5.1 BEAM #5

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **	(7) **
0	0	0	0		0	0	0
1	-4*	+14	+6	N.O.	+26	+61	-37
2	-4	62	20		40	76	-62
3	-8	120	58		60	170	-74
4	-10	199	111		72	217	-90
5	-13	290	176		88	291	-112
6	-16	406	258		112	352	-135
7	-20	492	328		124	434	-157
8	-22	573	458		156	535	-180
9	-26	652	597		183	696	-212
9.5	-23	728	684		206	788	-224
10.0	-26	796	786		225	926	-243
10.5	-25	880	1210		248	1044	-257
11.0	-29	917	1310		267	1158	-274
11.5	-26	979	1394		290	1365	-282
12.0	-28	1032	1472		310	1626	-294
12.5	-27	1166	1542		328	1792	-309
13.0	-30	1173	1596		354	1990	-319
13.5	-27	1241	1639		400	2125	-329
14.0	-29	1260	1674		465	2286	-343
14.5	-28	1342	1713		969	2430	-352
15.0	-29	1395	1764		1188	2586	-363
15.5	-29	1494	1784		1430	2716	-366
16.0	-30	1590	1827		1705	2878	-375
16.5	FAILURE						

* NOTE: + indicates tension, - indicates compression

** measurements in micro inch/inch

TABLE B.5.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(2)	(1)
i	0	0	0	0	0
ii	-3*	-2	+22	+57	+64
0	+21	+31	+66	+122	+41
1	+23	32	+67	121	+39
2	26	35	64	118	34
3	31	36	64	112	28
4	35	37	62	109	23
5	40	40	60	103	18
6	44	40	58	101	13
7	49	42	57	96	8
8	53	45	55	94	2
9	57	45	53	88	-4
9.5	58	45	53	85	-7
10.0	61	46	51	83	-13
10.5	64	44	48	78	-13
11.0	66	44	47	74	-14
11.5	73	42	46	23	-79
12.0	76	42	42	-13	-45
12.5	81	36	37	- 72	-129
13.0	88	34	32	- 102	-153
13.5	95	29	35	- 135	-188
14.0	99	27	21	- 167	-225
14.5	106	19	12	- 209	-280
15.0	113	18	2	- 256	-345
15.5	120	14	-7	- 306	-412
16.0	121	9	20	- 369	-493
16.5	FAILURE				

* NOTE: x 10^{-4} (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.5.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.02	.02
2	.03	,04	.03
3	.05	.06	.05
4	.07	.08	.07
5	.08	.10	.09
6	.10	.12	.11
7	.12	.14	.13
8	.14	.17	.15
9	.17	.19	.17
9.5	.18	.21	.20
10.0	.20	.23	.21
10.5	.21	.25	.22
11.0	.22	.26	.23
11.5	.23	.27	.25
12.0	.24	.29	.26
12.5	.25	.30	.27
13.0	.27	.32	.28
13.5	.28	.33	.30
14.0	.30	.35	.31
14.5	.32	.37	.33
15.0	.34	.39	.35
15.5	.36	.42	.37
16.0	.39	.45	.40
16.5	.42	.49	.43
17.0	.48	.57	.49
17.5	.58	.67	.58
18.0	.67	.78	.66
18.5	.77	.92	.78
19.0	.90	1.14	.90
19.5	1.00	1.26	1.00

* indicates 1/3 point

TABLE B.5.3 DEFLECTIONS

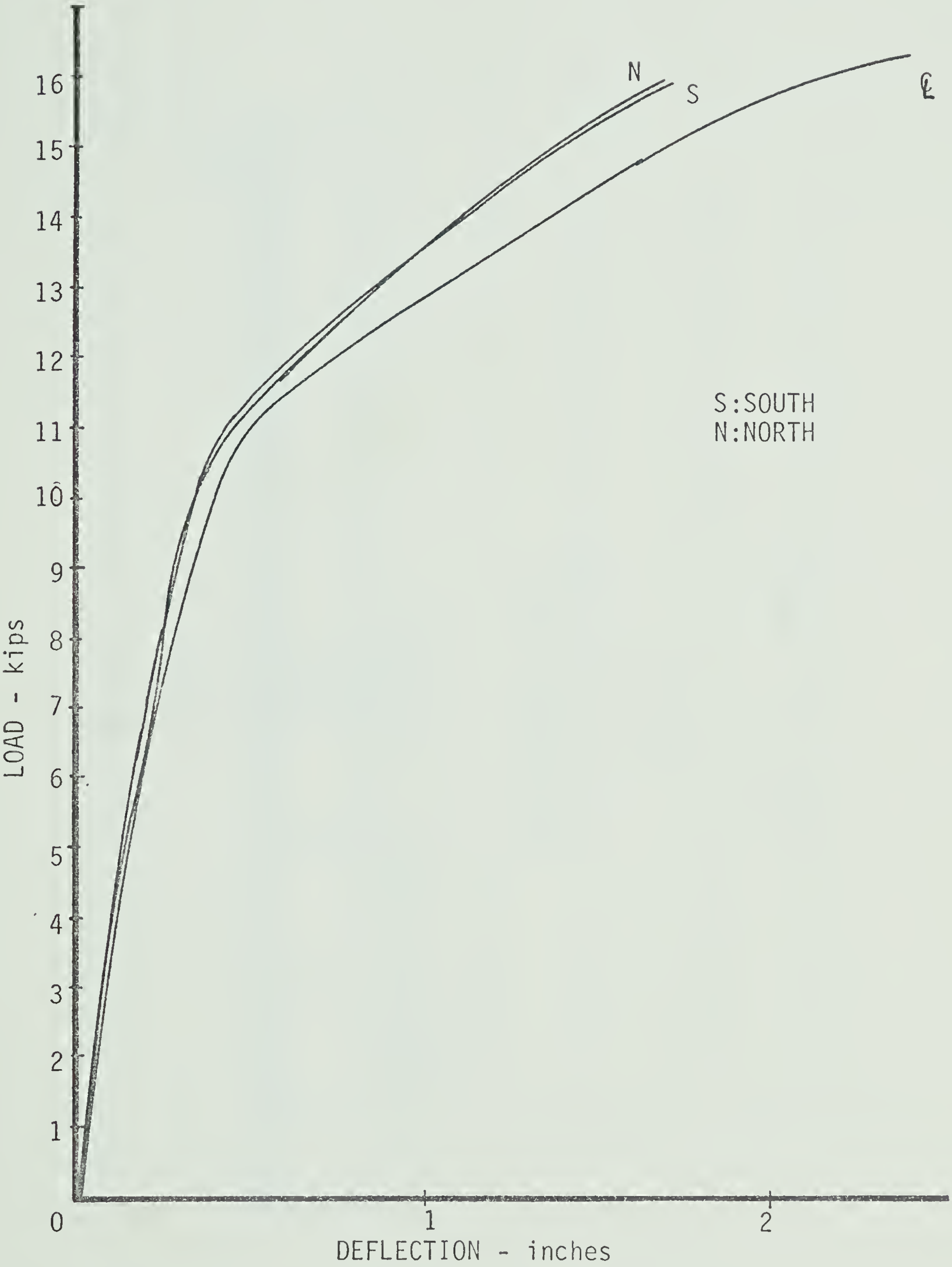
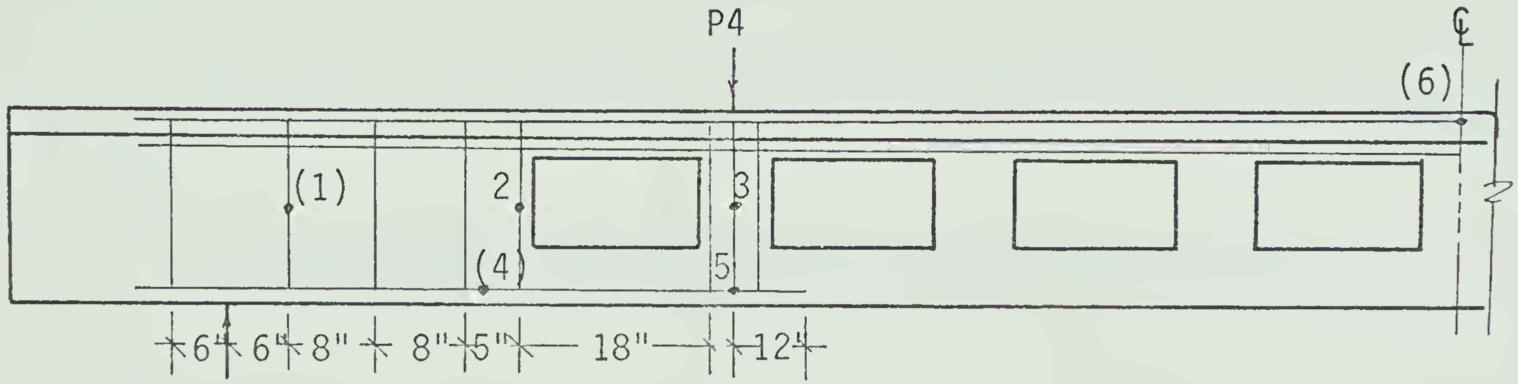


FIGURE B.5.2 LOAD-DEFLECTION DIAGRAM, BEAM #5



REINFORCEMENT DETAIL

STRAIN GAGE LOCATIONS

BEAM #6

FIGURE B.6.1 BEAM #6

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **
0	0	0	0	0	0	0
1	-6*	+7	-2	-1	+22	-34
2	-5	21	+3	+5	46	-70
3	-7	31	0	14	77	-97
4	-7	46	-3	16	100	-114
5	-19	71	-1	24	140	-132
6	-13	140	-6	30	165	-149
7	-16	191	-8	29	194	-169
8	-18	248	-8	15	217	-186
9	-21	270	-16	21	259	-214
9.5	-23	304	-6	20	282	-223
10.0	-24	320	-4	20	303	-230
10.5	-27	354	-4	15	327	-238
11.0	-30	370	-7	7	363	-246
11.5	-30	403	+3	9	422	-255
12.0	-32	421	1	-1	455	-271
12.5	-35	446	9	+10	560	-280
13.0	-29	474	13	5	655	-290
13.5	-32	510	20	-4	745	-295
14.0	-33	525	24	7	840	-305
14.5	-36	535	33	12	914	-316
15.0	-35	548	51	2	1011	-332
15.5	-37	562	60	3	1079	-343
16.0	-37	577	71	+6	1159	-351
16.5	-38	606	100	0	1237	-363
17.0	-38	630	115	17	1305	-378
17.5	-40	670	198	25	1365	-384
18.0	-42	700	485	36	1414	-408
18.5	-41	730	530	44	1490	-418
19.0	-41	751	547	62	1575	-423
19.5	-39	783	581	79	1707	-435

* NOTE: + indicates tension, - indicates compression

** Measurements in micro inch/inch

TABLE B.6.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(2)	(1)
i	0	0	0	0	0
ii	+5*	+30	+44	+78	+84
0	+16	+33	+76	+120	+132
1	+22	35	+76	118	+132
2	24	36	75	114	129
3	25	37	73	112	125
4	28	38	72	109	121
5	29	37	70	106	118
6	32	37	69	104	114
7	36	38	68	100	111
8	37	39	66	98	108
9	39	40	66	94	105
9.5	40	40	63	92	101
10.0	42	40	63	91	98
10.5	43	40	63	89	97
11.0	45	40	61	87	94
11.5	45	42	60	85	93
12.0	46	42	60	84	90
12.5	47	42	59	81	88
13.0	48	43	57	81	86
13.5	50	42	57	78	84
14.0	52	42	56	76	81
14.5	53	42	55	73	78
15.0	53	42	53	71	75
15.5	56	43	53	64	69
16.0	56	43	51	54	56
16.5	57	43	49	37	39
17.0	59	39	47	-31	18
17.5	63	40	47	- 79	-88
18.0	67	38	49	- 117	- 143
18.5	71	35	47	- 163	- 193
19.0	-----				
19.5	FAILURE				

* NOTE: x 10^{-4} inches (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.6.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.03	.05	.06
2	.06	.08	.09
3	.09	.11	.12
4	.11	.15	.15
5	.17	.22	.24
6	.20	.26	.27
7	.23	.29	.30
8	.26	.33	.33
9	.30	.38	.38
9.5	.35		[illegible]
10.0	.44	.43	.42
10.5	.51		
11.0	.59	.53	.51
11.5	.78	.63	.58
12.0	.87	.74	.67
12.5	.99	.97	.85
13.0	1.10	1.10	.95
13.5	1.23	1.23	1.07
14.0	1.38	1.39	1.18
14.5	1.56	1.54	1.31
15.0		1.74	1.47
15.5		1.97	----

* indicates 1/3 point

TABLE B.6.3 DEFLECTIONS

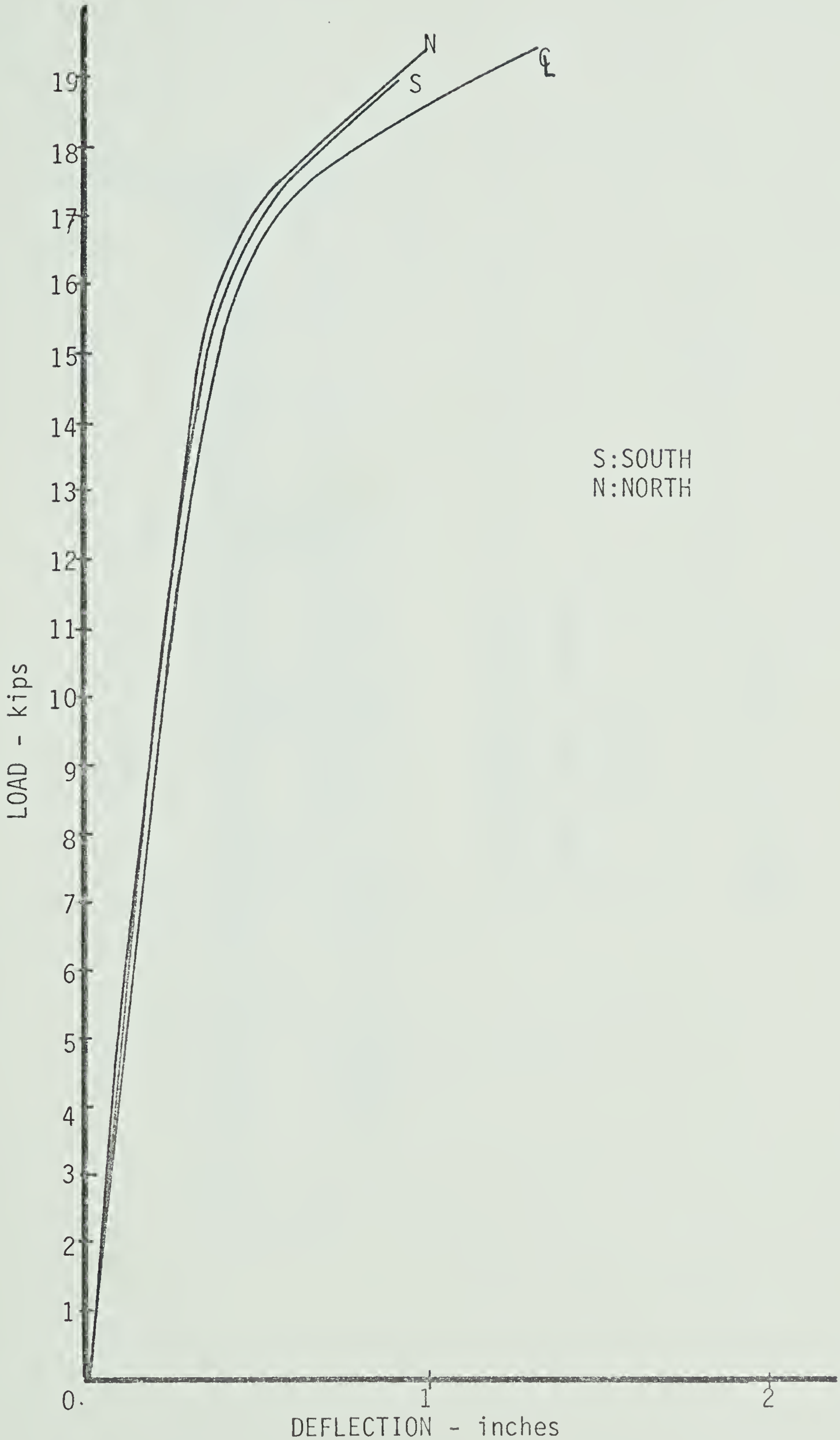
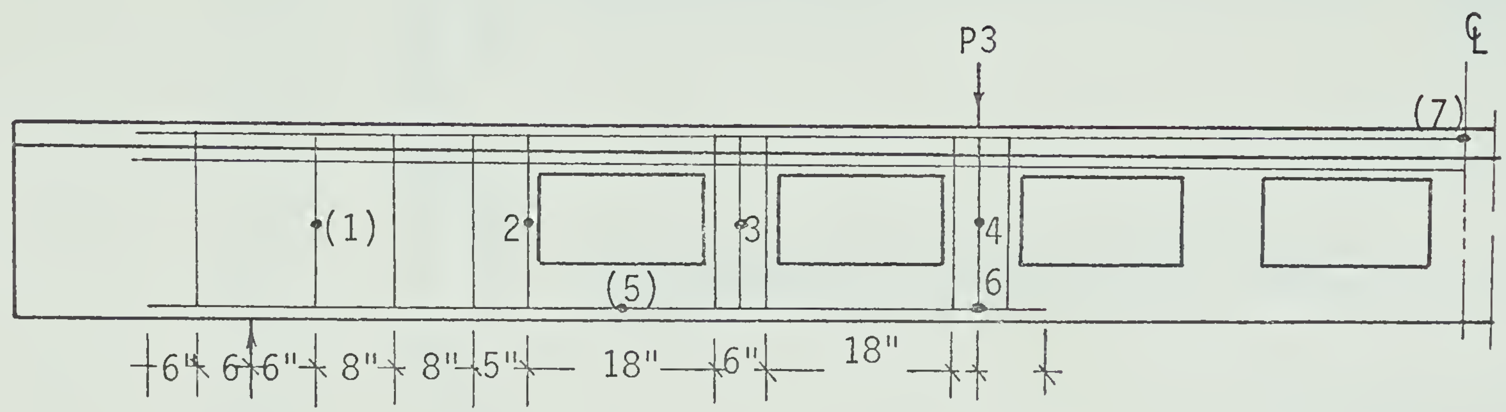


FIGURE B 6.2 LOAD-DEFLECTION DIAGRAM, BEAM #6



REINFORCEMENT DETAIL

STRAIN GAGE LOCATIONS

BEAM #7

FIGURE B.7.1 BEAM #7

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **	(7) **
0	N.O.	0	N.O.	0	N.O.	0	N.O.
1		0		0		-15	
2		+8*		-2		-22	
3		10		-4		-44	
4		7		0		-50	
5		126		-10		-70	
6		211		-2		-74	
7		257		-11		-94	
8		308		-2		-97	
9		357		-2		-117	
10		403		-+8		-122	
11		427		9		-135	
11.5		448		16		-137	
12.0		450		12		-148	
12.5		486		26		-152	
13.0		506		32		-157	
13.5		543		38		-160	
14.0		563		47		-166	
14.5		603		57		-167	
15.0		628		273		-172	
15.5		658		309		-175	

* NOTE: + indicates tension, - indicates compression

** Measurements in micro inch/inch

TABLE B.7.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(2)	(1)
i	0	0	0	0	0
ii	+2*	+9	+29	+70	+63
0	+19	+38	+64	+125	+132
1	+21	37	+63	122	+126
2	26	40	62	120	123
3	29	40	59	113	116
4	31	40	58	111	111
5	35	42	55	106	105
6	38	42	53	101	100
7	41	43	52	97	95
8	43	43	49	93	91
9	46	42	48	89	83
10	51	43	45	83	76
11	55	43	41	72	74
11.5	60	40	38	17	-6
12.0	63	35	28	-15	-51
12.5	71	32	19	-45	-74
13.0	76	33	10	-76	-106
13.5	82	35	-3	-120	-149
14.0	88	33	-15	-154	-167
14.5	93	34	-29	-198	-216
15.0	99	32	-46	-249	-264
15.5	106		--		----

* NOTE: x 10^{-4} inches (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.7.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.02	.02
2	.05	.06	.05
3	.08	.09	.08
4	.10	.12	.09
5	.13	.16	.14
6	.16	.19	.17
7	.19	.23	.20
8	.23	.27	.24
9	.27	.32	.28
9.5	.30	.35	.31
10.0	.34	.39	.34
10.5	.38	.43	.38
11.0	.42	.48	.43
11.5	.51	.62	.53
12.0	.60	.74	.63
12.5	.74	.90	.76
13.0	.85	1.05	.86
13.5	.97	1.19	.98
14.0	1.08	1.34	1.09
14.5	1.21	1.50	1.22
15.0	1.35	1.68	1.36
15.5	1.49	1.87	1.51
16.0	1.68	2.10	1.70
16.45	-----	2.38	

* indicates 1/3 point

TABLE B.7.3 DEFLECTIONS

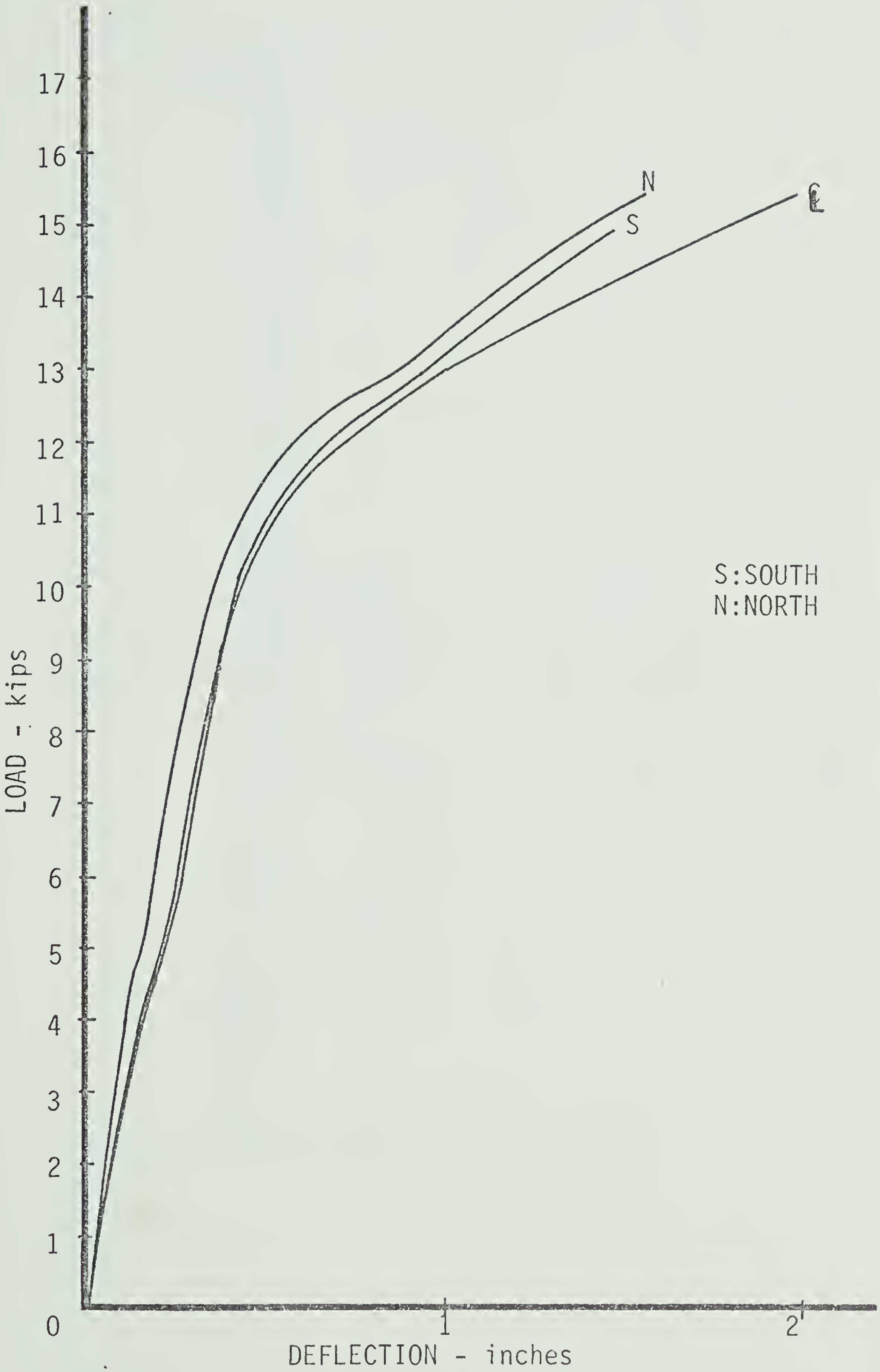
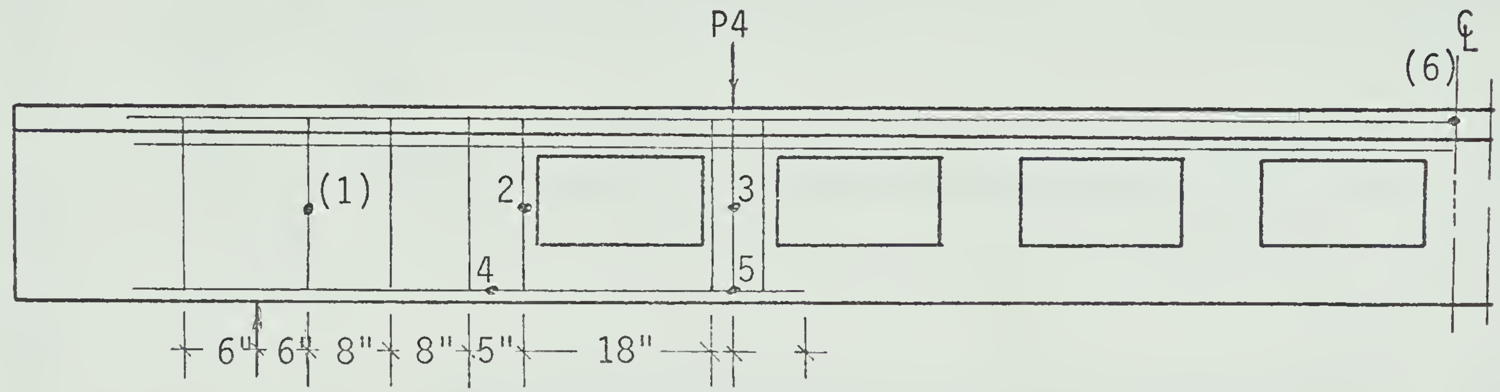


FIGURE B.7.2 LOAD DEFLECTION DIAGRAM, BEAM #7



REINFORCEMENT DETAIL

STRAIN GAGE LOCATIONS

BEAM #8

FIGURE B.8.1 BEAM #8

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **
1	-7*	0	-10	+4	N.O.	-10
2	-9	+4	-9	6		-19
3	-8	14	-10	6		-31
4	-10	35	-2	11		-41
5	-9	64	-14	11		-54
6	-11	84	-16	14		-62
7	-11	128	-18	11		-74
8	-14	176	-18	12		-84
9	-14	212	-18	11		-94
10.0	-16	258	-20	11		-103
11.0	-16	302	-18	12		-112
12.0	-16	334	-19	18		-119
13.0	-16	359	-16	24		-129
14.0	-16	379	-14	34		-134
15.0	-16	388	-8	41		-146
15.5	-16	398	-6	46		-149
16.0	-14	409	+4	58		-156
16.5	-16	438	239	66		-152
17.0	-14	454	264	76		-156
17.5	-15	484	292	88		-156
18.0	-14	506	311	94		-156
18.5	-12	513	316	103		-156
19.0	-12	528	322	114		-156
19.5	-12	536	331	121		-156
20.0	-12	548	342			-156
20.05	FAILURE					

* NOTE: + indicates tension, - indicates compression

** Measurements in micro inch/inch

TABLE B.8.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(2)	(1)
i	0	0	0	0	0
ii	+1*	+7	+30	+56	+66
0	25	25	76	123	137
1	26	26	76	120	132
2	28	26	74	117	129
3	30	27	73	114	127
4	33	28	73	114	124
5	35	28	72	111	121
6	37	29	71	109	118
7	40	30	70	107	115
8	43	31	68	104	112
9	45	32	67	101	108
10.0	47	33	67	98	105
11.0	49	34	64	95	103
12.0	51	36	62	92	98
13.0	54	35	63	89	94
14.0	56	35	60	84	88
15.0	57	37	59	82	86
15.5	59	39	59	80	83
16.0	61	37	56	75	83
16.5	66	33	53	73	86
17.0	68	32	55	71	83
17.5	70	31	53	68	82
18.0	74	29	52	69	82
18.5	76	27	53	66	82
19.0	78	24	53	65	82
19.5	80	23	28	-10	-24
20.o5	FAILURE				

*NOTE: X 10^{-4} inches (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.8.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.01	.02	.02
2	.03	.04	.03
3	.04	.05	.05
4	.06	.07	.06
5	.07	.09	.08
6	.09	.11	.09
7	.11	.13	.11
8	.13	.15	.13
9	.15	.17	.15
10.0	.17	.20	.17
11.0	.21	.22	.19
12.0	.24	.25	.22
13.0	.27	.28	.24
14.0	.30	.31	.27
15.0	.32	.34	.30
15.5	.34	.37	.32
16.0	.41	.39	.34
16,5	.49	.48	.41
17.0	.56	.58	.49
17.5	.73	.66	.56
18.0	.83	.83	.68
18.5	.95	.95	.80
19.0	1.05	1.09	.92
19.5	1.15	1.23	1.02
20.0		1.35	1.11

* indicates 1/3 point

TABLE B.8.3 DEFLECTIONS

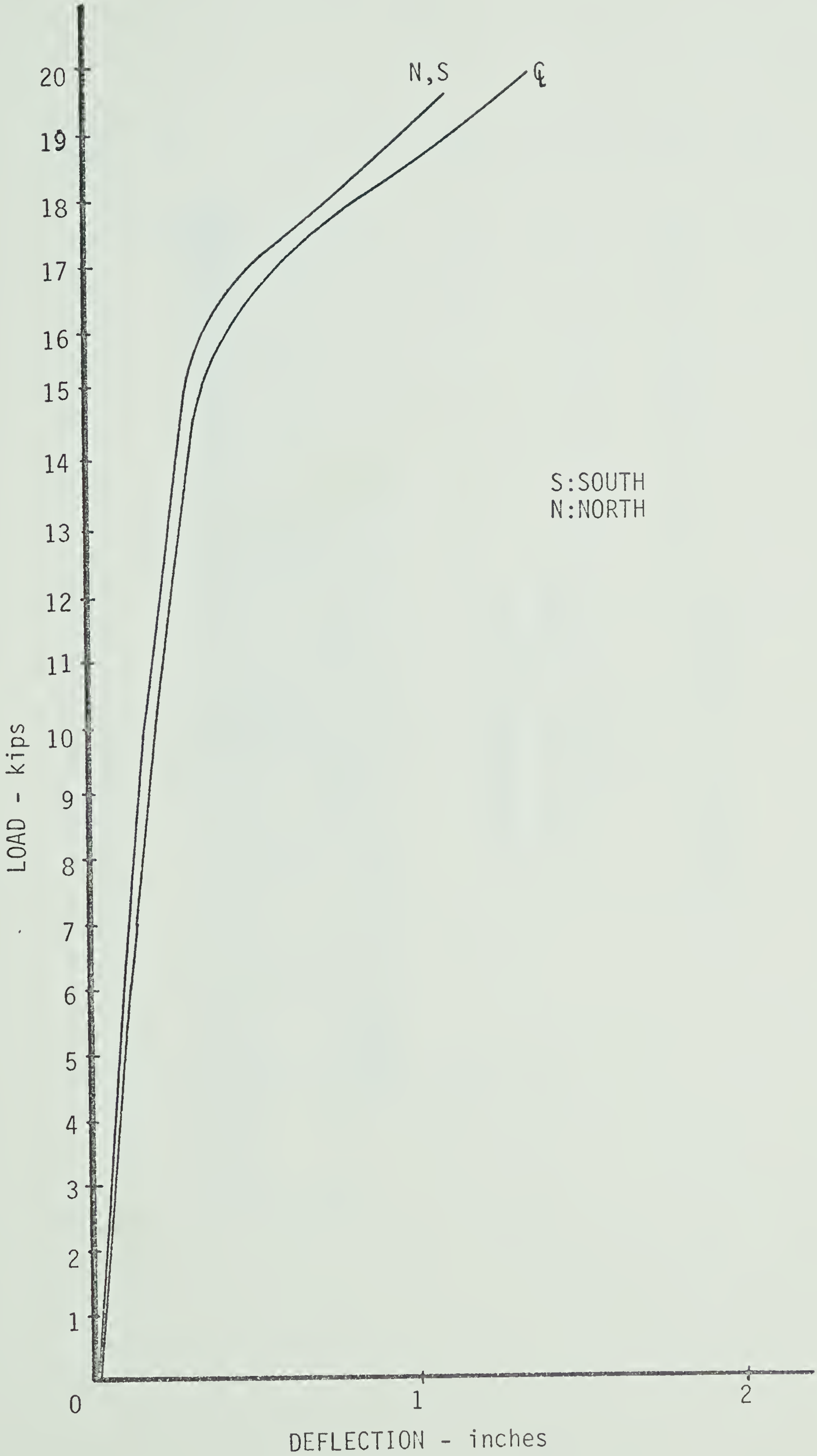
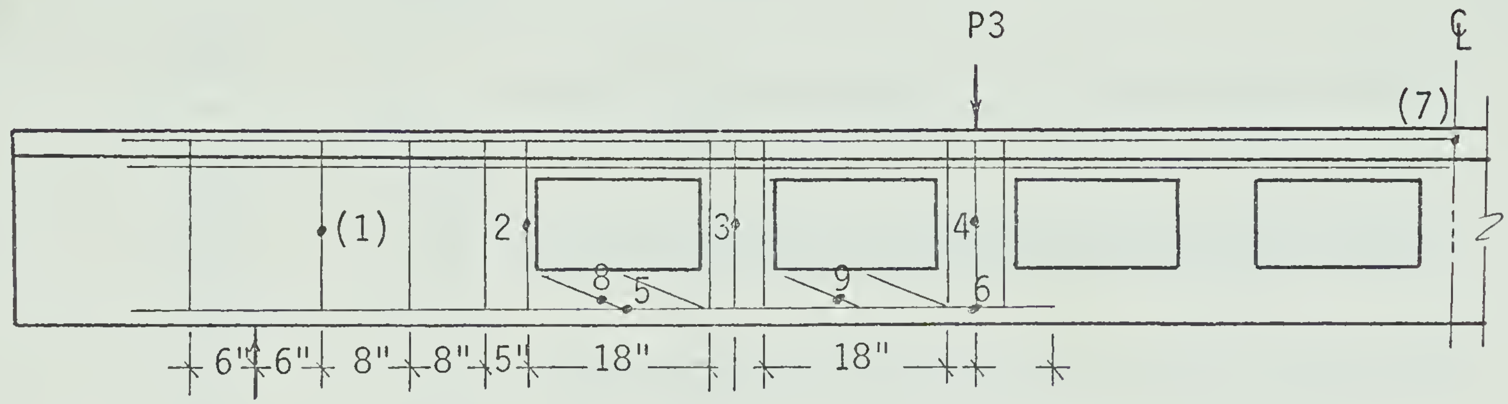


FIGURE B.8.2 LOAD-DEFLECTION DIAGRAM, BEAM #8



REINFORCEMENT DETAIL

STRAIN GAGE LOCATIONS

BEAM #9

FIGURE B.9.1 BEAM #9

LOAD (kips)	(1) **	(2) **	(3) **	(4) **	(5) **	(6) **	(7) **	(8) **	(9) **
1	-4*	+3	+1	-4	+11	+18	N.O.	+9	+6
2	-4	14	4	-9	2	38		9	10
3	-4	36	13	-4	4	61		14	11
4	-8	70	34	-4	46	85		11	16
5	-8	138	101	+4	61	111		16	18
6	-11	193	174	5	74	138		24	23
7	-12	238	234	14	92	165		29	26
8	-14	236	287	21	116	193		-91	32
9	-14	266	371	61	136	221		-81	43
10.0	-16	281	451	95	156	307		-75	61
11.0	-16	334	491	96	189	429		-61	34
11.5	-17	358	503	92	206	500		-52	31
12.0	-17	381	516	94	231	598		-39	31
12.5	-17	398	522	91	271	671		-29	24
13.0	-16	419	534	94	301	706		-16	21
13.5	-18	438	540	96	342	763		-6	14
14.0	-19	464	553	101	376	821		+6	16
14,5	-19	481	562	153	424	864		18	9
15.0	-19	509	576	181	476	821		39	4
15.5	-19	524	586	206	534	814		50	-2
16.0	-19	544	598	243	580	832		53	-6
16.5	-19	556	610	310	641	777		68	-16
17.0	-19	529	626	355	691	808		84	-20
17.5	-16	474	645		746	828		64	-28

* NOTE: + indicates tension, - indicates compression

** Measurements in micro inch/inch

TABLE B.9.1 STRAIN GAGE MEASUREMENTS

LOAD (kips)	(14)	(8)	(4)	(2(	(1)
i	0	0	0	0	0
ii	+5*	+7	+23	+54	+52
0	40	34	66	115	120
1	42	34	64	112	116
2	45	35	62	107	111
3	47	35	59	104	106
4	49	36	58	100	103
5	52	37	57	97	98
6	56	38	57	94	94
7	58	40	52	90	89
8	62	42	52	86	84
9	64	42	49	81	78
10.0	67	43	47	77	72
11.0	72	43	45	73	68
11.5	78	38	47	10	-30
12.0	82	35	44	-25	-72
12.5	88	32	45	-56	-110
13.0	92	31	41	-76	-139
13.5	100	29	42	-111	-184
14.0	105	28	39	-143	-221
14.5	116	27	37	-174	-260
15.0	121	25	31	-217	-303
15.5	125	24	23	-258	-346
16.0	132	23	14	-309	-394
16.5	141	20	0	-377	-456
17.0	152	18	-23	-460	-523
17.5	167	8	-48	-575	-528

* NOTE: X 10^{-4} inches (+ compression, - tension)

i indicates before release of strands
ii indicates after release of strands

TABLE B.9.2 DEMEC POINT MEASUREMENTS

LOAD (kips)	NORTH* (in.)	℄ (in.)	SOUTH* (in.)
1	.02	.02	.02
2	.04	.04	.04
3	.06	.07	.06
4	.08	.10	.08
5	.11	.13	.11
6	.14	.16	.14
7	.16	.19	.17
8	.19	.23	.20
9	.23	.27	.24
10.0	.28	.32	.28
11.0	.35	.41	.36
11.5	.43	.52	.45
12.0	.53	.64	.54
12.5	.65	.78	.65
13.0	.75	.92	.75
13.5	.86	1.06	.86
14.0	.98	1.21	.97
14.5	1.09	1.35	1.08
15.0	1.25	1.55	1.25
15.5	1.38	1.72	1.38
16.0	1.53	1.90	1.53
16,5	1.76	2.16	1.75
17.0	2.04	2.51	2.02
17.5	2.40	2.97	2.38

* indicates 1/3 point

TABLE B.9.3 DEFLECTIONS

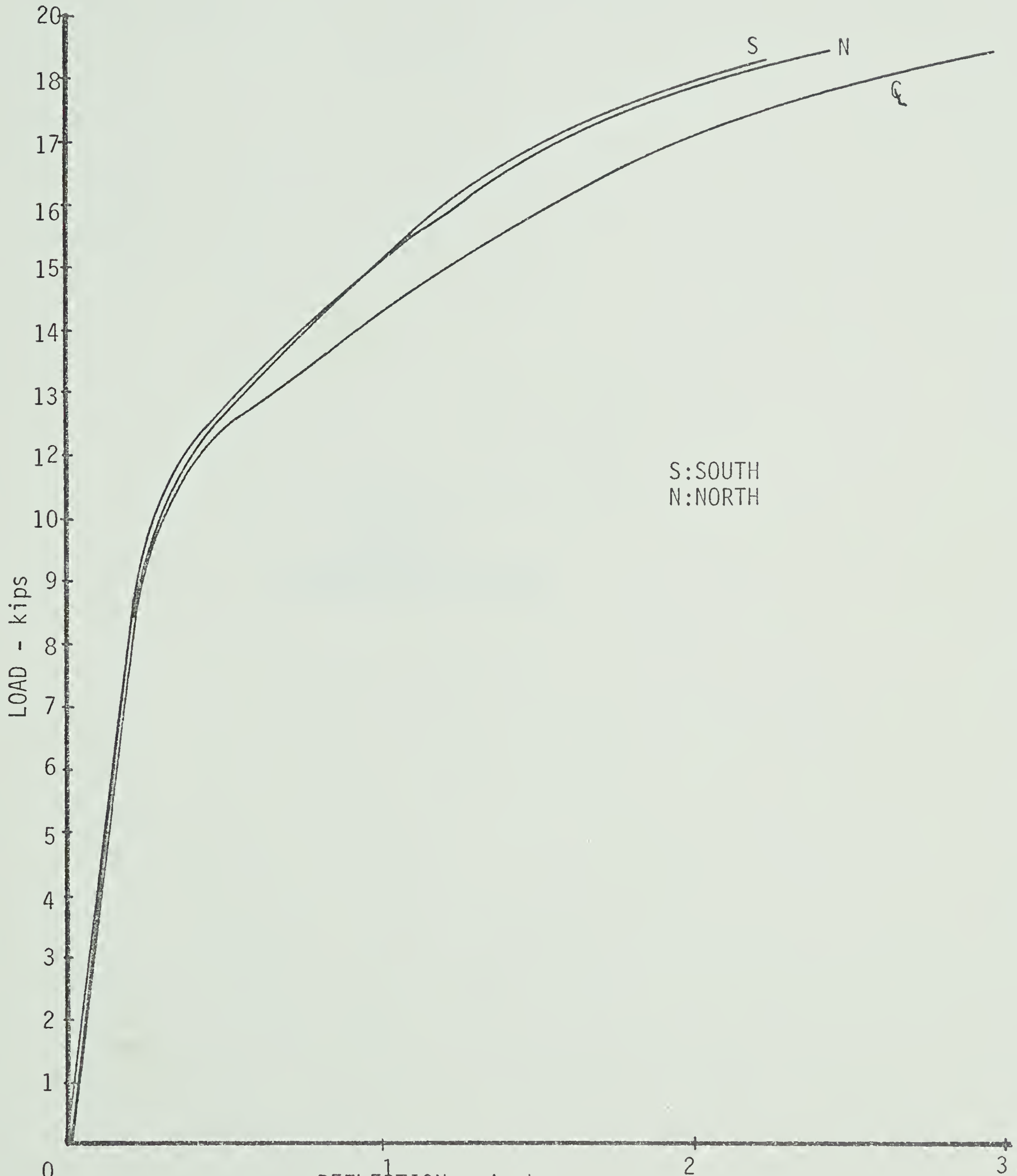


FIGURE B.9.2 LOAD-DEFLECTION DIAGRAM, BEAM # 9

APPENDIX C

NOTATION AND DESIGN

APPENDIX C

NOTATION

A_{ct} : Transformed area of section

A_s : Area of prestressing steel

I_t : Transformed moment of inertia

M_c : Dead load moment

M_{total}: Dead load moment + applied load moment

f'_c : Ultimate compressive strength of concrete

f'_s : Ultimate tensile strength of prestressing steel

f_i : Initial prestress

f_e : Effective prestress

P : Prestress force (effective)

y : Distance from geometric centroid to extreme fiber

e : Eccentricity of prestressing steel

k_t : Upper kern point

k_b : Lower kern point

a : Position of neutral axis at failure

f_{su} : Calculated ultimate steel stress

d : Effective depth

p : ratio of steel to concrete area

s : Stirrup spacing

Av : Area of stirrup

f_y : Yield stress of stirrup

V_u : Shear due to ultimate load

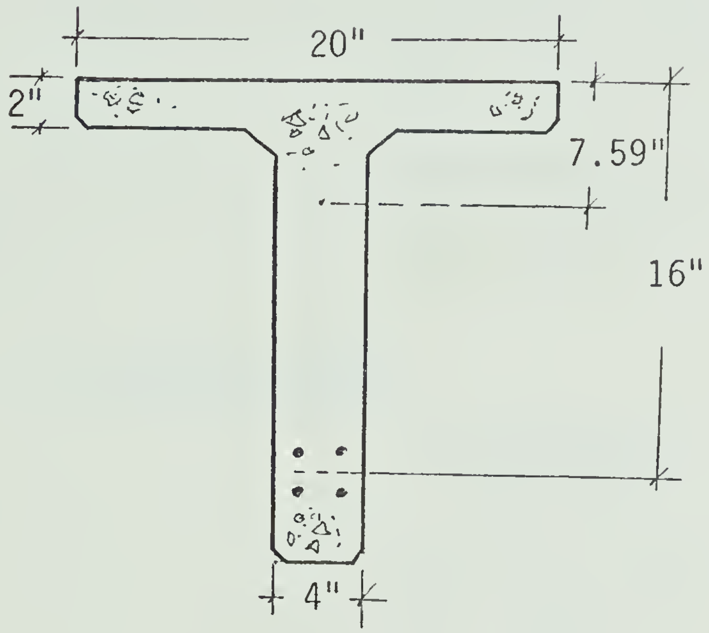
V_c : Shear carried by concrete

V_{ci} : Shear at diagonal cracking due to combined moment and shear

V_{cw} : Shear at diagonal cracking due to principal tension stresses in the web

APPENDIX C

DESIGN



$A_{ct} = 113.9 in^2$

$A_s = 0.3196 in^2$

$I_t = 4664 \ in^4$

$M_c = 70.5$ k-in.

$f'_c = 5000$ psi

$f'_s = 250$ ksi

$f_i = 150$ ksi

$f_e = 120$ ksi

(1) <u>Fiber stresses at transfer:</u>

Top at support: $f_c = 235$ psi tension

Top at ℄ : $f_c = 120$ psi tension

Bottom at ℄ : $f_c = 1305$psi compression

(2) <u>Kern points</u>:

$k_t = 3.26$ in.

$k_b = 5.34$ in.

(3) <u>Working load moment</u> (for "0" stress at ℄ bottom fibers):

$$M_{total} = \frac{I_t}{y}\left(\frac{P}{A_{ct}} + \frac{P \cdot e \cdot y}{I_t} \right)$$

$P = 120{,}000(0.3196) = 38{,}000$ lb.

$M_{total} = 450$ k-in.

$M_{applied} = 380$ k-in.

(4) Ultimate moment:

$a = 0.707$ in.

$$f_{su} = f'_s \left(1.0 - \frac{0.5pf'_s}{f'_c} \right) = 188 \text{ ksi}$$

$$M_u = A_s \cdot f_{su} \cdot d\ (1.0 - 0.59\ q)$$

$M_u = 938$ k-in.

(5) Ultimate shear:

$$s = \frac{A_v\ \phi\ d\ f_y}{V_u - \phi \cdot V_c}$$

$V_c = V_{ci}$ or V_{cw}

$V_{ci} = 7800$ lb.

$V_{cw} = 21{,}000$ lb.

Stirrup spacing

(A) No web openings:	1963 Code	1971 Code
$v_u = 361$ psi (1) Accounting for v_c ($v_c = \frac{V_{ci}}{b'd}$) V_{ci}=21,000lb.	s=19.6in.	s=19.6in.
(2) Minimum allowed $s = \frac{A_v(80)f_y\ d}{A_s\ f'_s} \left(\frac{b'}{d}\right)^{\frac{1}{2}}$	s=28.7in.	s=28.7in.
(3) Using $v_c = 1.7\ (f'_c)^{\frac{1}{2}} = 120$ psi	s=3.75in.	s=3.75in.
(4) Neglecting v_c (v_c=0)	s=2.5in.	s=2.5in.
(5) New code: $v_c = 0.6(f'_c)^{\frac{1}{2}} + \frac{700V_u d}{M_u}$ V_c=276 psi $5(f'_c)^{\frac{1}{2}}\ \ v_c\ \ 2(f'_c)^{\frac{1}{2}}$	//	s=10.6in.

(B) Web openings: (8" openings)

	1963 Code	1971 Code
$v_u = 722$ psi (1) $v_c = \frac{V_{ci}}{b'd} = 632$ psi	s=9.8in.	s=9.8in.
(2) Neglecting v_c ($v_c=0$)	s=1.25in.	s=1.25in.
(3) New code: $v_c=0.6(f'_c)^{\frac{1}{2}} = \frac{700V_ud}{M_u}$ $v_c = 276$ psi $(v_u - v_c) = 446$ psi	//	s=2.02in.